DPO Case File for DPO-2012-002

The following pdf represents a collection of documents associated with the submittal and disposition of a differing professional opinion (DPO) from an NRC employee involving the acceptability of AP1000 Shield Building.

Management Directive (MD) 10.159, "The NRC Differing Professional Opinions Program," describes the DPO Program. http://www.internal.nrc.gov/policy/directives/catalog/md10.159.pdf

The DPO Program is a formal process that allows employees and NRC contractors to have their differing views on established, mission-related issues considered by the highest level managers in their organizations, i.e., Office Directors and Regional Administrators. The process also provides managers with an independent, three-person review of the issue (one person chosen by the employee). After a decision is issued to an employee, he or she may appeal the decision to the Executive Director for Operations (EDO).

Because the disposition of a DPO represents a multi-step process, readers should view the records as a collection. In other words, reading a document in isolation will not provide the correct context for how this issue was considered by the NRC.

The records in this collection have been reviewed and approved for public dissemination.

Document 1: DPO Submittal
Document 2: Memo from Office Manager Establishing DPO Panel
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Document 8: Office Manager's Statement of Views
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Document 10: DPO Appeal Decision

Due to the volume and complexity of this case, a separate collection of DPO supporting documents was compiled and can be found in ADAMS, ML18338A309.

Westinghouse

Westinghouse Electric Company
Building 1, Suite 270A
1000 Westinghouse Drive
Cranberry Township, Pennsylvania 16066
USA

Mr. Robert Taylor
Division of New Reactor Licensing
Office of New Reactors

Direct tel: (412) 374-5093
e-mail: harperzs@westinghouse.com

Our ref: DCP_NRC_003324

July 03, 2018

Subject: Response to Request for Review of Differing Professional Opinion Documents Prior to Public Release Dated October 11, 2017

Dear Mr. Taylor:

By letter dated October 11, 2017, the Nuclear Regulatory Commission ("NRC") invited Westinghouse Electric Company LLC ("Westinghouse"), to identify proprietary information in documents associated with a differing professional opinion (DPO) regarding aspects of the AP1000 design certification.

Enclosed is the Westinghouse proprietary review of the first set of documents pursuant Title 10 of the Code of Federal Regulation (10 CFR) 2.390, "Public inspections, exemptions, requests for withholding."

Because the document contains proprietary information of Westinghouse, in conformance with the requirements of 10 CFR 2.390 of the NRC's regulations, we are enclosing an Application for Withholding Proprietary Information from Public Disclosure and an Affidavit. The Affidavit sets forth the basis on which the information identified as proprietary may be withheld from public disclosure by the Commission.

Redacted and marked versions of the document are attached to the enclosed Application for Withholding Proprietary Information from Public Disclosure and Affidavit. APP-GW-GLY-144, "Review of Differing Professional Opinion Documents Associated with the NRC Request Dated October 11, 2017 – Proprietary," and APP-GW-GLY-145, "Review of Differing Professional Opinion Documents Associated with the NRC Request Dated October 11, 2017 – Non-Proprietary," identifies the proprietary information in the Document.

Correspondence with respect to the proprietary aspects of the Application for Withholding or the Westinghouse Affidavit should reference AW-18-4767, and should be addressed to me at the above address.

Zachary S. Harper
Manager, AP1000 Licensing

Westinghouse

Westinghouse Electric Company
1000 Westinghouse Drive
Cranberry Township, Pennsylvania 16066
USA

U.S. Nuclear Regulatory Commission
Document Control Desk
11555 Rockville Pike
Rockville, MD 20852-2738

Direct tel: (412)-374-4372
e-mail: monohajs@westinghouse.com

AW-18-4767

July 03, 2018

APPLICATION FOR WITHHOLDING PROPRIETARY
INFORMATION FROM PUBLIC DISCLOSURE

Subject: Response to Request for Review of Differing Professional Opinion Documents Prior to Public Release Dated October 11, 2017

Reference: Letter from Zachary S. Harper to R. Taylor, DCP_NRC_003324, dated July 03, 2018

The Application for Withholding Proprietary Information from Public Disclosure is submitted by Westinghouse Electric Company LLC ("Westinghouse"), pursuant to the provisions of paragraph (b)(1) of Section 2.390 of the Nuclear Regulatory Commission's ("Commission's") regulations. It contains commercial strategic information proprietary to Westinghouse and customarily held in confidence.

The proprietary information for which withholding is being requested in the above-referenced report is further identified in Affidavit AW-18-4767 signed by the owner of the proprietary information, Westinghouse. The Affidavit, which accompanies this letter, sets forth the basis on which the information may be withheld from public disclosure by the Commission and addresses with specificity the considerations listed in paragraph (b)(4) of 10 CFR Section 2.390 of the Commission's regulations.

Correspondence with respect to the proprietary aspects of the Application for Withholding or the accompanying Affidavit should reference AW-18-4767, and should be addressed to James A. Gresham, Manager, Regulatory Compliance, Westinghouse Electric Company, 1000 Westinghouse Drive, Building 2, Suite 259, Cranberry Township, Pennsylvania 16066.

Very truly yours,

Jill S Monahan

Jill S. Monahan, Manager

Licensing Inspection & Special Programs

Westinghouse Electric Company
1000 Westinghouse Drive
Building 1, Suite 262D
Cranberry Township, Pennsylvania 16066
USA

Enclosures to AW-18-4767

1. AFFIDAVIT
2. PROPRIETARY INFORMATION NOTICE and COPYRIGHT NOTICE
3. APP-GW-GLY-144, Revision 0, "Review of Differing Professional Opinion Documents Associated with the NRC Request Dated October 11, 2017 – Proprietary"
4. APP-GW-GLY-145, Revision 0, "Review of Differing Professional Opinion Documents Associated with the NRC Request Dated October 11, 2017 – Non-Proprietary"

ENCLOSURE 1 to AW-18-4767

AFFIDAVIT

AFFIDAVIT

COMMONWEALTH OF PENNSYLVANIA:

ss

COUNTY OF BUTLER:

I, Jill S. Monahan, am authorized to execute this Affidavit on behalf of Westinghouse Electric Company LLC ("Westinghouse") and declare that the averments of fact set forth in this Affidavit are true and correct to the best of my knowledge, information, and belief.

Executed on: 7-3-2018

Jill S Monahan

Jill S. Monahan, Manager
Licensing Inspections and Special Programs

(1) I am Manager, Licensing Inspections and Special Programs, Westinghouse Electric Company LLC (“Westinghouse”), and as such, I have been specifically delegated the function of reviewing the proprietary information sought to be withheld from public disclosure in connection with nuclear power plant licensing and rule making proceedings, and am authorized to apply for its withholding on behalf of Westinghouse.

(2) I am making this Affidavit in conformance with the provisions of 10 CFR Section 2.390 of the Nuclear Regulatory Commission’s (“Commission’s”) regulations and in conjunction with the Westinghouse Application for Withholding Proprietary Information from Public Disclosure accompanying this Affidavit.

(3) I have personal knowledge of the criteria and procedures utilized by Westinghouse in designating information as a trade secret, privileged or as confidential commercial or financial information.

(4) Pursuant to the provisions of paragraph (b)(4) of Section 2.390 of the Commission’s regulations, the following is furnished for consideration by the Commission in determining whether the information sought to be withheld from public disclosure should be withheld.

(i) The information sought to be withheld from public disclosure is owned and has been held in confidence by Westinghouse.

(ii) The information is of a type customarily held in confidence by Westinghouse and not customarily disclosed to the public. Westinghouse has a rational basis for determining the types of information customarily held in confidence by it and, in that connection, utilizes a system to determine when and whether to hold certain types of information in confidence. The application of that system and the substance of that system constitute Westinghouse policy and provide the rational basis required.

Under that system, information is held in confidence if it falls in one or more of several types, the release of which might result in the loss of an existing or potential competitive advantage, as follows:

(a) The information reveals the distinguishing aspects of a process (or component, structure, tool, method, etc.) where prevention of its use by any of

Westinghouse's competitors without license from Westinghouse constitutes a competitive economic advantage over other companies.

(b) It consists of supporting data, including test data, relative to a process (or component, structure, tool, method, etc.), the application of which data secures a competitive economic advantage, e.g., by optimization or improved marketability.

(c) Its use by a competitor would reduce his expenditure of resources or improve his competitive position in the design, manufacture, shipment, installation, assurance of quality, or licensing a similar product.

(d) It reveals cost or price information, production capacities, budget levels, or commercial strategies of Westinghouse, its customers or suppliers.

(e) It reveals aspects of past, present, or future Westinghouse or customer funded development plans and programs of potential commercial value to Westinghouse.

(f) It contains patentable ideas, for which patent protection may be desirable.

(iii) There are sound policy reasons behind the Westinghouse system which include the following:

(a) The use of such information by Westinghouse gives Westinghouse a competitive advantage over its competitors. It is, therefore, withheld from disclosure to protect the Westinghouse competitive position.

(b) It is information that is marketable in many ways. The extent to which such information is available to competitors diminishes the Westinghouse ability to sell products and services involving the use of the information.

(c) Use by our competitor would put Westinghouse at a competitive disadvantage by reducing his expenditure of resources at our expense.

(d) Each component of proprietary information pertinent to a particular competitive advantage is potentially as valuable as the total competitive advantage. If competitors acquire components of proprietary information, any one component may be the key to the entire puzzle, thereby depriving Westinghouse of a competitive advantage.

(e) Unrestricted disclosure would jeopardize the position of prominence of Westinghouse in the world market, and thereby give a market advantage to the competition of those countries.

(f) The Westinghouse capacity to invest corporate assets in research and development depends upon the success in obtaining and maintaining a competitive advantage.

(iv) The information is being transmitted to the Commission in confidence and, under the provisions of 10 CFR Section 2.390, it is to be received in confidence by the Commission.

(v) The information sought to be protected is not available in public sources or available information has not been previously employed in the same original manner or method to the best of our knowledge and belief.

(vi) As described in Westinghouse Letter DCP_NRC_003324, the proprietary information sought to be withheld in this submittal is that which is appropriately marked and contained in Westinghouse Document No. APP-GW-GLY-144, Revision 0, “Review of Differing Professional Opinion Documents Associated with the NRC Request Dated October 11, 2017 – Proprietary”. The proprietary information is that associated with NRC request for review of differing professional opinion documents prior to public release dated October 11, 2017.

(a) This information is part of that which will enable Westinghouse to support AP1000 new reactor projects and other customer projects, including its processes to design and obtain licensing approvals for the AP1000 shield building.

(b) Further this information has substantial commercial value as follows:

(i) Westinghouse plans to sell the use of similar information to its customers for the purpose of plant construction and operation.

(ii) Westinghouse can sell support and defense of industry guidelines and acceptance criteria for plant-specific applications.

(iii) The information requested to be withheld reveals the distinguishing aspects of a methodology which was developed by Westinghouse.

Public disclosure of this proprietary information is likely to cause substantial harm to the competitive position of Westinghouse because it would enhance the ability of competitors to provide similar plant safety systems and licensing defense services for commercial power reactors without commensurate expenses. Also, public disclosure of the information would enable others to use the information to meet NRC requirements for licensing documentation without purchasing the right to use the information.

The development of the technology described in part by the information is the result of applying the results of many years of experience in an intensive Westinghouse effort and the expenditure of a considerable sum of money.

In order for competitors of Westinghouse to duplicate this information, similar technical programs would have to be performed and a significant manpower effort, having the requisite talent and experience, would have to be expended.

Further the deponent sayeth not.

ENCLOSURE 2 to AW-18-4767

PROPRIETARY INFORMATION NOTICE and COPYRIGHT NOTICE

PROPRIETARY INFORMATION NOTICE

Transmitted herewith are proprietary and non-proprietary versions of a document, furnished to the NRC in connection with requests for generic and/or plant-specific review and approval.

In order to conform to the requirements of 10 CFR 2.390 of the Commission's regulations concerning the protection of proprietary information so submitted to the NRC, the information which is proprietary in the proprietary versions is contained within brackets, and where the proprietary information has been deleted in the non-proprietary versions, only the brackets remain (the information that was contained within the brackets in the proprietary versions having been deleted). The justification for claiming the information so designated as proprietary is indicated in both versions by means of lower case letters (a) through (f) located as a superscript immediately following the brackets enclosing each item of information being identified as proprietary or in the margin opposite such information. These lower case letters refer to the types of information Westinghouse customarily holds in confidence identified in Sections (4)(ii)(a) through (4)(ii)(f) of the Affidavit accompanying this transmittal pursuant to 10 CFR 2.390(b)(1).

COPYRIGHT NOTICE

ENCLOSURE 4 to AW-18-4767

APP-GW-GLY-145, Revision 0, "Review of Differing Professional Opinion Documents Associated with the NRC Request Dated October 11, 2017 – Non-Proprietary"

DPO Case File for DPO-2012-002

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The DPO Program is a formal process that allows employees and NRC contractors to have their differing views on established, mission-related issues considered by the highest level managers in their organizations, i.e., Office Directors and Regional Administrators. The process also provides managers with an independent, three-person review of the issue (one person chosen by the employee). After a decision is issued to an employee, he or she may appeal the decision to the Executive Director for Operations (EDO).

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Due to the volume and complexity of this case, a separate collection of DPO supporting documents was compiled and can be found in ADAMS, ML18338A309.

Document 1: DPO Submittal

NRC FORM 680
(11-2002)
NRCMD 10.159

U.S. NUCLEAR REGULATORY COMMISSION

DIFFERING PROFESSIONAL OPINION

FOR PROCESSING USE ONLY

1. DPO CASE NUMBER: DPO-2012-002

2. DATE RECEIVED: 7/6/2012

INSTRUCTIONS: *Prepare this form legibly and submit three copies to the address provided in Block 14 below.*

3. NAME OF SUBMITTER	4. POSITION TITLE	5. GRADE
John S. Ma	Senior Structural Engineer	GG-15

6. OFFICE/DIVISION/BRANCH/SECTION	7. BUILDING	8. MAIL STOP	9. SUPERVISOR
NRO/DE/SEB1	TWFN	10H9	[redacted]

10. DESCRIBE THE PRESENT SITUATION, CONDITION, METHOD, ETC., WHICH YOU BELIEVE SHOULD BE CHANGED OR IMPROVED
(Continue on Page 2 or 3 as necessary.)

The certified AP1000 shield building is unsafe, because (1) the certified AP 1000 shield building does not meet the NRC's seismic margin requirement, (2) there is no adequate demonstration that the shield build meets the GDC 2 requirement, (3) the NRC's conclusion that the aircraft missile would not penetrate the AP 1000 shield building wall is illogical, and (4) the shield building wall is insufficiently strong and ductile to resist earthquakes or aircraft missiles.

Additional details are provided in the attached discussion.

11. DESCRIBE YOUR DIFFERING OPINION IN ACCORDANCE WITH THE GUIDANCE PRESENTED IN NRC MANAGEMENT DIRECTIVE 10.159.
(Continue on Page 2 or 3 as necessary.)

The AP1000 shield building should be redesigned such that it clearly meets all regulatory requirements. In order to meet the GDC 2 requirement, the NRC seismic margin requirement, and the aircraft missile impact requirement, design modifications to the shield building are required. The required modifications include the increase of the wall thickness and ductility, and a redesign of the connections between the roof and the water tank.

Additional details are provided in the attached discussion.

12. Check (a) or (b) as appropriate:

- [x] a. Thorough discussions of the issue(s) raised in item 11 have taken place within my management chain; or
- [ ] b. The reasons why I cannot approach my immediate chain of command are:

SIGNATURE OF SUBMITTER	DATE	SIGNATURE OF CO-SUBMITTER *(if any)*	DATE
John S. Ma	7/05/2012		

13. PROPOSED PANEL MEMBERS ARE *(in priority order)*
1. Dave Jeng
2. [redacted]
3. Jerry Chung

14. Submit this form to:
Differing Professional Opinions Program Manager
Office of: Enforcement
Mail Stop: O-4A15A

15. ACKNOWLEDGMENT

THANK YOU FOR YOUR DIFFERING PROFESSIONAL OPINION. It will be carefully considered by a panel of experts in accordance with the provisions of NRCMD 10.159, and you will be advised of any action taken. Your interest in improving NRC operations is appreciated.

SIGNATURE OF DIFFERING PROFESSIONAL OPINIONS PROGRAM MANAGER (DPOPM): Renee Pedersen

PRE-CONDITIONS MET [x] YES [ ] NO

DATE OF ACKNOWLEDGMENT: 7/11/2012

THE CERTIFIED AP1000 SHIELD BUILDING IS UNSAFE
by
John S. Ma, Ph.D., P. E.
June 29, 2012
Senior Structural Engineer and Charter Member of NRC
Member of ACI and ASCE
Recipient of ACI Raymond C. Reese Structural Research Award Medal
Registered (Licensed) Professional Engineer in State of Maryland, Virginal, and California

The AP 1000 shield building provides protection for the steel containment from exterior missiles, such as aircraft missiles, and cooling function to the containment during the reactor accident inside the containment, and its collapse would most likely crash the steel containment, due to the heavy weight of the cooling water tank located on top of the shield building roof, and potentially damage reactor cores and create uncontrolled radiation release. Therefore, the shield building design is not only required to be an impact- and earthquake-resistant structure, but is also required to demonstrate that it possess a high confidence and low failure probability in accordance with the NRC's seismic margin requirement. A new type of concrete element, which ACI Code design equations do not apply, was proposed by Westinghouse as building blocks to form the shield building wall. The Code requires the design of the new concrete element to be verified by testing as to its suitability for use, and, requires the entire wall to be designed in accordance with the Code's procedures, criteria, and requirements. The NRC, assisted by two world leading authorities in seismic design for concrete shell types of structures, met with Westinghouse for several days to jointly establish acceptance criteria and a testing program for judging the suitability on the design of the new concrete element. After the proposed designed concrete element failed in a brittle manner, and failed the acceptance criteria, during testing, the NRC accepted the brittle concrete element as building blocks for the wall despite the author and the two experts insisted that the brittle concrete element should be rejected, and a new design for the concrete element started. Westinghouse submitted analysis results by a nonlinear static pushover computer code as the demonstration of safe design for the entire wall in place of the design that is required to be performed in accordance with ACI Code's procedures, criteria, and requirements. The NRC also accepted the pushover analysis method and its results and the wall design as being safe despite the author's objection to the inapplicability of the pushover analysis method, and its use as a replacement to the ACI Code's method, for the safety evaluation of the shield building. The NRC' acceptance of the brittle concrete element and the entire wall designed by the pushover analysis method not only prematurely short circuited the ongoing review process but also prematurely released Westinghouse from performing unfinished analyses that are necessary to the safety review based on the ACI Code. The author has demonstrated that (1) the certified AP 1000 shield building does not meet the NRC's seismic margin requirement, (2) there is no adequate demonstration that the shield building meet the 10 CFR Part 50, Appendix A, General Design Criterion (GDC) 2 requirement, (3) the NRC's conclusion that the aircraft missile would not penetrate the AP 1000 shield building wall is illogical, and (4) the shield building wall is insufficiently strong and ductile to resist earthquakes or aircraft missiles. The causes for the unsafe design are (1) that the misuse of brittle concrete wall element as building blocks to form the cylindrical shell type of wall for missile barriers as well as for the seismic load resistant system of the shield building that supports a heavy water tank on top of its roof violates the basic principle of physics and structural engineering is improper, (2) that the misuse of a <u>static</u> pushover analysis method to predict <u>dynamic (seismic)</u> behavior of the shield building, and as the method in place of the ACI Code's method for the design of the entire shield building wall is improper, (3) that Westinghouse, an NSSS vendor, is not equipped to perform aircraft missile impact analysis, and thus its analysis results not only contradicts with that of ESBWR, which was performed by an outside reputable structural engineering firm specialized in

the missile impact field, but also contradicts with concrete structural engineering principles and test data because the brittle concrete wall has little punching shear strength and deformability against the punching shear force and energy generated by the aircraft missile, and (4) that the NRC's inaction to the author's and other staff's concerns on the inadequacy of the shield building subjected to aircraft missiles and Westinghouse analysis is improper. As a practicing structural engineer, an NRC charter member, and the ex-lead reviewer for the AP 1000 shield building subjected to earthquakes, the author considers that it is his responsibility and obligation, to report the unsafe design and hopes that the design will be modified to become a safe design so that the shield building can protect the safety, health, and welfare of the public. In order to meet the GDC 2 requirement, the NRC seismic margin requirement, and the aircraft missile impact requirement, design modifications to the shield building are required. The required modifications include the increase of the wall thickness and ductility, and a redesign of the connections between the roof and the water tank. Without modifications to the design, the certified AP 1000 shield building, which is weak and brittle, will become sitting nuclear bombs, at different locations in the world, waiting for explosion, when they are triggered by aircraft missiles, or earthquakes.

INTRODUCTION

The AP 1000 shield building has a cylindrical shell type of walls like ordinary reinforced concrete cylindrical shell type of walls. However, the concrete element that forms the wall is different from standard reinforced concrete components. The word "element" is also referred in the ACI Code as "section," in the AP 1000 shield building as "module," and "component" by the NRC. An element, a section, a module, or a component, is a reference to the geometrical arrangement of steel reinforcement and concrete within the thickness of the concrete wall. In ordinary reinforced concrete structures, the concrete element is formed by steel reinforcing bars fully embedded in and surrounded by concrete. The behaviors of these concrete elements and structures under different types of loading are known through testing. The ACI Code design procedures, criteria, and requirements, are derived from these test data for these ordinary reinforced concrete elements and structures. However, unlike this conventional configuration, the concrete element for the AP1000 shield building is formed by using two exterior steel faceplates and filled with concrete between the plates.

The NRC rejected the original design of the concrete element that had no steel "tie-bars" between the faceplates (ML092320205). Westinghouse provided steel tie bars in a later design. This type of concrete element differs from that of the ordinary reinforced concrete elements in that the interior steel reinforcing bars in ordinary reinforced concrete are replaced by exterior steel plates for the AP 1000 shield building wall. ACI Code's design equations are not applicable to this new type of concrete elements. Therefore, the Code requires testing to demonstrate the suitability and adequacy of the design for new types of concrete elements.

A dome shape of roof, built with concrete supported by steel beams, is connected with, and supported by, the cylindrical shell type of walls. A heavy concrete water tank sits on, and is anchored to, the top of the shield building roof, and the water provides cooling to the steel containment during the reactor accident. If portion of the shield building wall collapses, or the roof fails, the heavy water tank would most likely crash the steel containment, which is below it, and potentially create core damage and uncontrolled radiation release. Because of the possibility of creating containment and core damages, the AP 1000 shield building design is required to demonstrate that it meets the NRC's seismic margin requirement.

The author was the lead reviewer for the adequacy of the AP 1000 shield building subjected to earthquakes, but was not on the adequacy of the shield building subjected to missiles, and has reasons to believe that the shield building was not properly analyzed for aircraft missiles by Westinghouse, and has repeatedly raised concerns on it to the NRC. The reasons for the concerns are provided and explained later.

The review on the adequacy of the shield building includes the supporting cylindrical wall, the roof, and the water tank. A proper review of the adequacy of the shield building wall relies upon confirmation of the adequacy of the concrete wall element. However, the NRC made a decision to accept the inadequate brittle wall element and the entire AP 1000 shield building design under the objection of the author. The review process leading to the NRC's acceptance of the shield building can be summarized as follows:

- <u>Original design was clearly inadequate</u>: The main reason for the NRC to reject the Westinghouse's original design of the concrete wall element was that the concrete element, with no steel "tie-bar" to connect the faceplates, would not work as a unit and had no ductility.
- <u>NRC and Westinghouse agreed on tests and test acceptance criteria to demonstrate adequacy of revised design</u>: The NRC staff and its two consultants, who are experts in seismic design for concrete shell type of structures, met with Westinghouse for several days, and reached an agreement on the approach and acceptance criteria for the design of this new type of concrete elements for the AP 1000 shield building wall. The acceptance criteria are that the element must be ductile and possess a ductility ratio (definition of ductility ratio is provided and explained later) of [ ] [a,c] in both the out-of-plane or radial (perpendicular to the wall thickness) direction and the in-plane (tangential to the circumferential) direction. The approach is that the ductility ratio must be verified by testing. The approach and acceptance criteria used for the new element of the AP 1000 shield building are similar to that had been used for ordinary reinforced concrete elements. For example, a ductility ratio of **four** is required in New Zealand because of its high seismic activities (see the consultants' letters in references 1 and 2). A cyclic test for a ductility ratio of **four** for an ordinary reinforced element, which was tested by the current ACI president and was published in 1975, is shown in the text later as a comparison to the ductility of the Westinghouse's concrete element test.
- <u>Revised design failed tests</u>: The concrete element with widely spaced "tie-bar" reinforcement failed in a brittle manner with no ductility during the out-of-plane shear test and the concrete element for the in-plane shear test was incorrectly built and improperly tested and the test was stopped before ductility ratio of one was reached, and both test concrete elements were far from being meeting the ductility acceptance criteria.
- <u>Westinghouse proposed a static computer analysis in lieu of testing and ACI Code's analysis and design requirements</u>: After the tests failed to meet the acceptance criteria, Westinghouse submitted a nonlinear <u>static</u> pushover analysis results as the justification

for the safe design of the shield building under seismic (dynamic) loads to replace the agreed-upon testing approach and criteria for the new type of concrete elements and other procedures, criteria, and requirements in the ACI Code for the analysis and design of the entire shield building wall.

- NRC improperly accepted analysis, neglecting test results: While the two NRC consultants and the author insisted that the design of the concrete element resulted in brittle failure should not be accepted, and the justification on the adequacy of the design for new concrete elements must be demonstrated through testing, the NRC decided to accept the design of the concrete element with brittle failure, and the Westinghouse's static analysis results as the justification for a safe design, and ignored the testing approach and acceptance criteria that were established and agreed upon between the NRC and Westinghouse, and other requirements in the ACI Code for the analysis and design of the entire shield building wall.
- NRC ignored its staffs' concerns on inadequate design for the shield building subjected to aircraft missile impact: The author and the technical staff, who was in charge of AIA program, have raised concerns on the inadequacy of the shield building design subjected to aircraft missiles, but the concerns were ignored.

The NRC's decision to accept the brittle concrete element and the nonlinear static pushover analysis results as the justification for a safe design not only short circuited the ongoing review process, which was based on the ACI Code's procedures, criteria, and requirements, but also released Westinghouse from performing Code required analyses that without their results safety of the shield building would be difficult to assess, as will be shown later.

Each of the following NRC actions is incorrect with respect to structural engineering practices, and the combination of these incorrect actions caused the certified AP 1000 shield building to be unsafe, and is unprecedented to regulatory agencies that in charge of building safety in the modern structural engineering history known to the author.

- Accepting brittle concrete elements as basic building blocks to form the shield building wall that its primary function is to resist missile impact violates the fundamental structural engineering principle with respect to impact design, and is against impact test data
- Misinterpreting ACI Code's ductility requirements for concrete elements for seismic design as being "no requirement" is inconceivable and detrimental to its evaluation on seismic Category I concrete structures
- Arbitrarily exercising its will to accept the brittle concrete element with no technical merit by trashing the acceptance approach and criteria for the design of concrete elements, which was jointly established between the NRC and Westinghouse, with assistance from two world leading authorities in seismic design, is incomprehensible
- Ignoring and neglecting ACI Code's procedures, criteria, and requirements, which were established and agreed by the structural engineering profession and endorsed by the NRC, for the design and review of concrete buildings, not only deviates from NRC's Standard Review Plans and Regulatory Guides but also from normal practices by structural engineers in building designs or reviews

- Accepting the nonlinear <u>static</u> pushover analysis method as the tool to predict <u>seismic (dynamic)</u> behaviors of the shield building during earthquakes is incredible, especially the pushover analysis method is prohibited by building codes for use on an irregular, or tall, or safety important, building while the shield building is irregular and tall and safety important
- Accepting the nonlinear static pushover analysis results in lieu of the required analysis results from ACI Code's procedures, criteria, and requirements as the demonstration of safe design is improper and unpreedented

The author performed a review on the safety of the shield building in accordance with structural engineering principles, and the philosophy and requirements of the NRC-endorsed ACI Code, demonstrating (1) the certified AP 1000 shield building does not meet the NRC's seismic margin requirement, (2) there is no adequate demonstration that the shield building meet the 10 CFR Part 50, Appendix A, General Design Criterion (GDC) 2 requirement, (3) the NRC's conclusion that the aircraft missile would not penetrate the AP 1000 shield building wall is illogical, and (4) the shield building wall is insufficiently strong and ductile to resist earthquakes or aircraft missiles. Therefore, the shield building design is inadequate and unsafe, and design modifications to the inadequate design are required in order to meet the NRC's seismic margin requirement, GDC 2 requirement, and aircraft missile impact requirement.

REQUIREMENTS FOR SAFE DESIGN AND REGULATORY SAFETY REVIEW

In order to achieve a safe design, the design structural engineer must understand (I) structural engineering principles, which describe structural behaviors under applied loads, and (II) the philosophy and requirements of building codes, which provide **<u>minimum</u>** requirements that structures must meet to avoid structural failure or collapse. The associated regulatory safety review is to confirm that structures have been properly designed. Therefore, a regulatory reviewer should also possess similar knowledge and experience as the design structural engineer.

I. Structural Engineering Principles

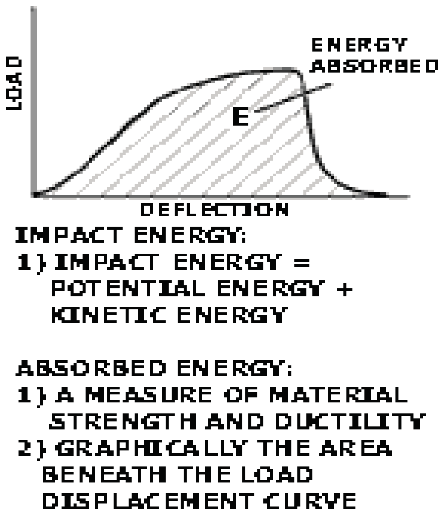
Structural engineering principles are based on natural laws of physics, empirical knowledge from laboratory testing, and field observations of structures during or after earthquakes, tornado/hurricane winds, or missile impact. Structural engineering principles require that a concrete element/structure to possess sufficient strength to resist applied forces (stresses), sufficient ductility to deform in the direction of missile impact, or cyclically during earthquakes, and sufficient energy absorption capability to balance or resist the energy generated by missile impact, or sufficient energy dissipation capability to dissipate energy generated by earthquake ground motion. Violation of these principles could result in unsafe design of structures.

I.1 Structural Engineering Principles for Missile Barrier Design

A missile barrier must

- use ductile material/element
- will not shatter under the impact force
- possess energy absorption capability greater than the impact energy generated by the missile
- use laboratory test method to determine the energy absorption capability of an element/structure, or the ACI Code method to calculate the energy absorption capability for an element/structure with known load-deflection (resistance-displacement) curves from test data.

An excerpt from (www.instron.com/wa/applications/**test**_types/**impact**/default.aspx) testing laboratory on the behavior of materials/elements subjected to impact loading and the amount of energy an element/structure can absorb are provided below.



"Brittle materials take little energy to start a crack, little more to propagate it to a shattering climax. Other materials possess ductility to varying degrees. Highly ductile materials fail by puncture in drop weight testing and require a high energy load to initiate and propagate the crack."

ACI 349-06 "Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary," states "If the energy balance method is used, only the energy represented by Area A in Fig. RF.8, (shown below) which is available to resist the impulsive and impactive loads, should be used," as shown below.

The area "A" consists of an area of a triangle bounded by a vertical line at Xy, which represents the yield point of the element and a slant line, and a rectangular area bounded by Xy and Xm, which is defined by Xy multiplies the ductility ratio (definition of ductility ratio will be provided later), Ud, and represents the energy absorption capability due to the ductility of the element. As can be seen in the diagram, without ductility, the triangular area is only a fraction of the rectangular area, and is insufficient to resist energy imparted to it by a high energy missile impact.

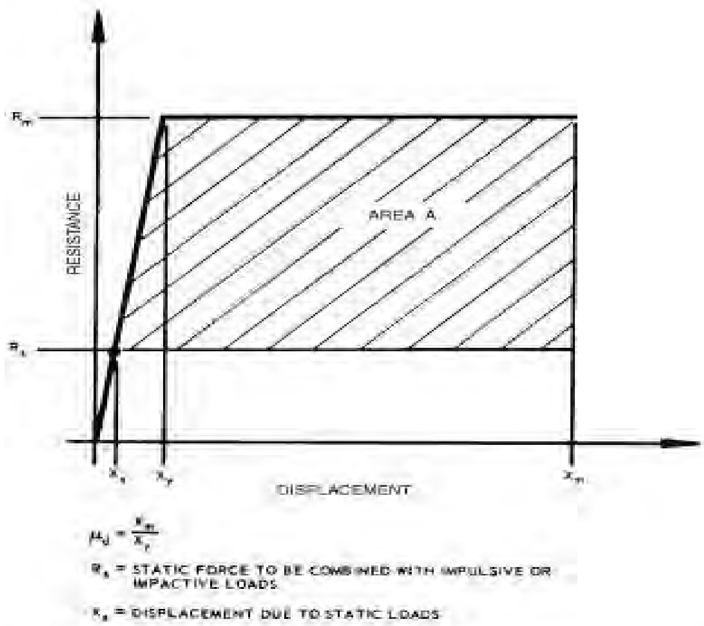


Fig. RF.8—Available resistance: idealized resistance-displacement curve.

Besides the energy absorption capability consideration, **the shattering effect**, as stated in the impact testing above, is another reason that a brittle element/structure should not be used for missile barriers. This shattering effect can be understood by the following example. An adult can stand on a board which is placed on top of a wine glass and will not break the glass because the glass has strong compressive strength. However, tapping the wine glass with a small rod fast enough, the wine glass will shatter into pieces due to its brittleness.

Results from impact testing and structural engineering theories on energy balance principles and the shattering effect have concluded that for missile barriers only ductile element/structures should be used, and brittle element/structures should be prohibited.

I.2 Structural Engineering Principles for Seismic Design

An element/structure subjected to seismic loads must

- possess sufficient strength to resist applied forces
- possess sufficient ductility to deform cyclically without failure
- possess sufficient energy dissipation capability to dissipate the energy imparted to it from earthquake ground motions

When the ground moves back and forth during earthquakes, the foundation of a structure moves with it. However, the mass and its weight of different circumferential layers of the shield building in the vertical direction above the foundation tends to stay at its original idled locations, and thus shear forces are generated between layers in the vertical direction of the building because the bottom layer tends to move but the top layer tends to stay idle. These cyclic shear forces and shear deformations become the dominant contributors to the failure or collapse of building structures during earthquakes. Therefore, in order to avoid building failure or collapse, the shear strength, shear ductility, and shear energy dissipation capability of elements in the shield building should be designed to be greater than the corresponding shear forces, cyclic deformations, and the energy imparted to it, due to ground motions, in accordance with the ACI Code requirements.

The definition of shear strength, shear ductility, and shear energy dissipation capability is provided below. A comparison of values of shear strength, shear ductility, and shear energy dissipation capability between the AP1000 brittle concrete element and an almost identical ordinary reinforced concrete element with respect to the shear strength index (see definition below) are provided below to show the vast difference between the brittle concrete element of the AP 1000 shield building and the ductile ordinary concrete element resulting from different types of construction techniques and tie-bar materials although both elements having the [ ] [a,c]

a,c

Note: The reinforced concrete module (element) in blue color did not fail at the ductility ratio (definition provided below) of four (4) at the first cycle. The element was tested cyclically to simulate a required cyclic displacement, generated by earthquake ground motion, equals to four times the displacement, as shown in the right diagram above, at the time the steel started yielding. When the displacement reached a ductility ratio of four (4), the displacement loading was reversed to the opposite direction. The element failed after twelve (12) cycles vs. [ ] [a,c] of a cycle for the AP1000 concrete element. The ordinary reinforced concrete element had a shear reinforcement index pFy=0.16 vs. the shear reinforcement index pFy= [ ] a,c for the AP1000 test concrete element, which are [ ] a,c p represents the percentage of steel area of tie-bars within an element, and Fy the yield strength of the tie-bars. Shear reinforcement index is the major indication of shear strength of concrete elements. The cyclic test of the ordinary reinforced concrete element is documented in reference 3.

Strength (How Strong is the element/structure?):

- The highest point on the Load–Deflection (ductility ratio in this drawing) curve, which is a measure of strength (capacity) against applied load (force) to the element/structure.

Ductility (How deformable is the element/structure?):

- Ductility is generally represented by displacement ductility ratio, and the displacement ductility ratio is calculated by the displacement of an element/structure at failure divided by the displacement of the element/structure when steel starts yielding. The higher the displacement ductility ratio, the more ductile the element/structure is. Ductility is a measure of deformability of the element/structure without failure.

Energy absorption/dissipation capability (How tough is the element/structure):

- The energy absorption/dissipation capability is measured by the area between the curve and the abscissa (blue lines or red lines). The greater the area, the more energy absorption/dissipation capability the element/structure possesses. This is the most important characteristic or material property of an element/structure that indicates its capability to resist exterior energy imparted to it from earthquake ground motion or from missile impact, such as aircraft missiles.

I.3 AP1000 Shield Building Design Violates Structural Engineering Principles

The drawing above clearly demonstrates that the AP1000 shield building concrete element is brittle, has no cyclic deformability in out-of-plane shear, and possesses little energy dissipation/absorption capability in out-of-plane shear, and thus should not have been approved for a building that is supposed to be an impact-resistant and earthquake-resistant structure. The NRC should never accept a concrete element/structure that violates the basic principle of structural engineering. However, the NRC did. As a result of defying the law of physics and the principle of structural engineering, the unsafe design and use of the brittle element as building blocks to the shield building wall will naturally lead to an unsafe AP 1000 shield building.

II. The Philosophy and Requirements of Building Codes

The purpose of building codes is to provide **minimum** requirements to prevent building collapse and thus safeguard the public safety, health, and general welfare, and prescribe analysis and design procedures, criteria, and requirements, to prevent structural engineers from neglecting necessary steps for completing a safe design or review. The philosophy and requirements of building codes, such as the International Building Code, or ACI Code, for the design of earthquake-resistant and impact-resistant structures are based on meeting the structural engineering principles with respect to specific requirements on **strength**, **ductility**, and the **energy absorption/dissipation capability** of a concrete element/structure. The Code's approach is to demonstrate that the applied force, which is referred to as "demand," to an element/structure should be less than the strength, which is referred to as "capacity" of the element/structure, and that the same demonstration for ductility and energy absorption/dissipation capability to an element/structure be performed.

Based on ACI Code's procedures, criteria, and requirements, which are endorsed by building codes in the United States and the NRC, the author compares the "demand" of the in-plane shear force in the certified AP 1000 shield building wall generated by earthquakes to the "capacity" of the certified AP 1000 shield building wall as an example to show the inadequacy of the certified AP 1000 shield building wall design.

II.1 Strength (Capacity)

II.1.1 ACI Code's Maximum (Legal) Value for In-Plane Shear Strength

If an element/structure can sustain combined forces (axial force in tension or compression, in-plane shear force, out-of-plane shear force, and torsion), which are applied to it, without being crashed, torn apart, sheared off, or twisted off, that is a demonstration of sufficient strength of the element/structure. The strength or "ultimate strength" or "capacity" of an element is specified in the ACI Code, based on laboratory test data. The ACI Code's specified ultimate value for in-plane shear strength is 10 square root of the ultimate compressive strength of concrete ($\mathbf{10\sqrt{f_c'}}$), meaning that the calculated in-plane shear stress should not exceed this legal limit of shear strength.

II.1.2 Recommended Maximum Value for Design for Moderately Tall Wall Buildings

It is recognized that the in-plane shear strength in walls reduces as the height of the wall increases. In a paper entitled, "Design and Detailing of Moderately Tall Wall Buildings," by Professor Jack P. Moehle, the Director of the Earthquake Engineering Research Center at UC Berkeley, California, it is stated "Because walls are primary lateral load resisting elements of a structural system, it should not be the designer's aim to make a wall as thin as practicable, but rather to construct it to be as sturdy as necessary. Experimental studies (Aktan and Bertero; 1985) indicate that seemingly brittle failure modes in the laboratory are possible due to web crushing when nominal shear stresses are high. With this aspect in mind, it is recommended that the maximum nominal shear stress in a wall under maximum expected shears should not exceed approximately $\mathbf{6\sqrt{f_c'}}$ psi" (see reference 4). The relevant page of reference 4 is attached. Professors Bertero and Moehle are recognized authorities in seismic concrete structure design, and their recommendations are well respected and received by structural engineers.

II.1.3 In-Plane Shear Strength Reduced due to Presence of Out-Of-Plane Shear Force

Due to the unique feature of the cylindrical shape of the AP1000 shield building wall, an element in the wall is subjected to both in-plane and out-of-plane shear forces simultaneously during earthquakes. However, the ACI Code's specified ultimate value for in-plane shear strength, which equals to10 square root of the ultimate compressive strength of concrete ($10\sqrt{f_c'}$), was obtained from test data of elements subjected to in-plane shear force alone without the effect of out-of-plane shear force. This is because most test equipments (machines) can only apply one directional shear force to an element.

There is only one test machine in the world that can apply both in-plane and out-of plane shear forces simultaneously to concrete elements, which is located in the Thomas Hsu Laboratory at the University of Houston. The research results on the strength of concrete elements influenced by the interaction of in-plane and out-of-plane shear forces are shown below, and will be published later in a book entitled "Infrastructure Systems for Nuclear Energy," by Thomas Hsu, and others, by John Wily and Sons. The test data clearly indicated that the in-plane shear strength (capacity) was reduced due to the effect of the presence of out-of-plane shear force. These interaction effects have not yet been codified into the ACI Code because the data are new. Had this interaction effect been considered, the in-plane shear strength should have been reduced and the blue "capacity" line in the drawing of in-plane shear force vs. ductility ratio, as shown later in Section II.1.5, should have been shifted toward the left.

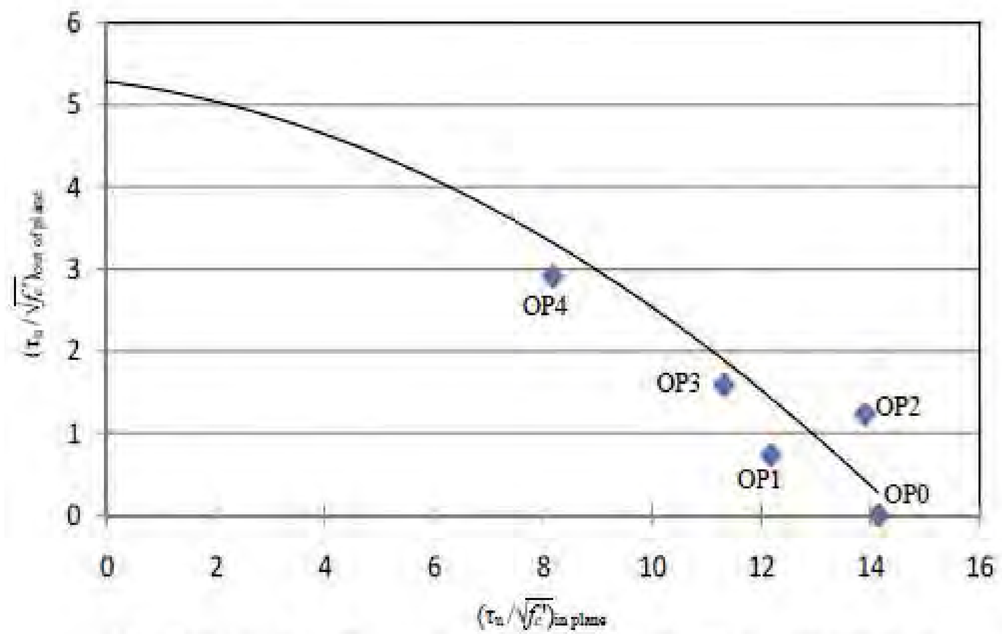


Figure 15. Interaction Diagram between In-Plane and Out-of-Plane Shear Strength

II.1.4 Demand (Applied Force)

ACI 349 "Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary," which is endorsed by the NRC, requires that seismic Category I structures, which includes the AP 1000 shield building, should be designed to function linear elastically under the design basis earthquake, which is also called safe shutdown earthquake (SSE). To function linear elastically means that the shear force generated by SSE in the element/structure should be limited in the linear line, which connects from the origin to the point of ductility ratio equals to one (1.0 Dy) as shown in the diagram in Section I.2. This design philosophy and requirement is to ensure that the element/structure will not suffer major damage, except some minor concrete cracking, during and after the SSE. The demand is the "actual" in-plane shear force (stress) applied to the element/structure. The word "actual" represents the most probable value or magnitude after proper considerations. As a result of the NRC's decision to prematurely accept the shield building design as being safe, Westinghouse was prematurely released from performing analyses that are necessary to obtain the "actual" forces (axial tension or compression, in-plane shear, out-of plane shear, and torsion) in the elements of the shield building wall. Therefore, the demand of forces as calculated and presented by Westinghouse are less than the actual demand of forces that are required for the design, or for use to compare with the strength (capacity), of the shield building, as explained below.

II.1.4.1 Calculated Applied In-Plane Shear Force Is Less than the Actual Force

II.1.4.1.1 Irregular Structure and Improper Modeling

The shield building with the brittle concrete element has an irregular zigzag or step shape at the bottom, as shown in the drawing below (the final shape may be slightly different from this preliminary draft drawing), and is anchored to concrete in a zigzag shape both in the circumferential and meridional directions. Furthermore, the auxiliary building roof is connected to portions of the shield building wall that is not shown in the drawing, which exerts forces into the shield building wall during earthquakes. Therefore, the AP 1000 shield building is a highly irregular structure due to its boundary (support) conditions.

Building codes and research results have shown that irregular shapes of structures create difficulties for adequate analytical methods and thus affect the reliability of analysis results. After irregular structures failed during the 1994 Northridge earthquake, the 1997 Uniform Building Code prohibited the use of highly irregular structures in near-fault regions.

Due to the irregular supporting conditions, torsion will be generated in the shied building during earthquakes. This type of torsion is called “inherent torsion”, which is generated due to unsymmetrical geometries of a structure either in the horizontal (circumferential) direction or in the vertical (meridional) direction. In the AP1000 shield building case, the structure is restrained by unsymmetrical anchors in both circumferential and meridonal directions (axes). Therefore significant torsional forces (effects) exist during earthquakes. These torsional forces generate in-plane-shear forces in the shield building wall. However, these in-plane-shear forces or torsional forces are vanished in the Westinghouse’s mathematical model because a perfect cylindrical wall was assumed for the shield building without the zigzag boundary conditions.

a,c

II.1.4.1.2 Code Required Accidental Torsion Analysis Is Not Performed

Building codes, such as the International Building Code and the ACI Code, require accidental torsion analysis. The NRC Standard Review Plan (SRP), Section 3.7.2, II, item11, also requires accidental torsion analysis. The accidental torsion accounts for many things, such as the uneven density of concrete exists in different locations of the structure due to concrete placements or out-of-round shape due to practical construction deviations as opposed to a perfect cylindrical shape of the assumed mathematical model of the shield building. The accidental torsion analysis will generate shearing force (stress) in the plane of circular sections of the cylindrical walls, and is thus named “in-plane shear.” However, Westinghouse did not perform the accidental torsion analysis. As a result of no accidental torsion analysis for the AP1000 shield building, the calculated in-plane shear force (stress) in the cylindrical wall is less than the actual shear force in elements of the shield building wall. Neglecting the Code required accidental torsion analysis is a gross negligence.

II.1.4.1.3 Code Required P-Delta Effect Analysis Is Not Performed

Analysis and design of a structure or a wall require an iterative process. The process starts with a linear elastic analysis (first order) of a mathematical model of geometry and boundary (restrain or support) conditions, with assumed dimensions (height and thickness of the wall), and nominal or elastic (no cracking is assumed in concrete) wall stiffness subjected to applied loads, such as winds or earthquakes. The results of the first order linear elastic analysis will yield forces (axial, shear, and torsion) in elements of the wall, and horizontal deformation (deflection) of elements (sections) along elevations (heights) of the wall. However, these resulting forces and deformations from the first order linear elastic analysis take no account of the additional effect due to the deformations of the wall under winds or earthquakes. This is because it was not possible to know the amount of deformations of a wall under applied loads, such as winds or earthquakes, prior to the analysis, and thus the mathematical formulation of the first order linear elastic analysis did not include the effect of horizontal deformations as a result of applied loads. Therefore, ACI Code requires a second order analysis, which is also called P-Delta analysis (P represents the vertical weight, or load, on the top of a wall element, and Delta represents the horizontal deformation of elements (sections) of the wall at different elevations (heights) during earthquakes or winds. This second order P-Delta analysis yields additional forces to the wall elements. These extra forces are then added to those forces obtained from the first order analysis to become the actual forces at wall elements (sections) due to the applied loads. These actual forces are combined with other forces generated from other appropriate sources, such as the accidental torsion, to become the final design forces for wall elements (sections). The design of wall elements is to ensure that these elements possess strength (capacity) greater than the required design final forces (demand) in accordance with ACI Code’s procedures, criteria, and requirements. If the strength (capacity) of wall elements is less than the final design forces (demand), then the wall thickness would have to be increased and a new analysis and design cycle restarts until the strength (capacity) of wall elements is greater than the final design forces (demand). This is a typical analysis and design process for nuclear power plant, as well as conventional, structures because structural engineering principles and ACI Code require the safety of structures to be demonstrated through such a process.

However, Westinghouse did not perform the second order P-Delta analysis, and therefore these extra forces due to the P-Delta effect are not accounted for in the final design forces. These forces generated by the P-Delta effect on wall sections are directly proportional to the magnitude of the P and Delta values. The vertical load, P, on the wall includes the heavy weight of the PCCWST and water inside the tank, and thus increases the P-Delta effect significantly more than ordinary walls that do not have such a heavy weight on top of them. During the second order P-Delta analysis, the nominal or elastic stiffness of wall elements assumed in the first order analysis is reduced to account for the concrete cracking due to wall element deformations generated by winds or earthquakes. The less stiff wall would yield greater wall deformations (Delta) than that of the first order analysis that had greater wall stiffness because no concrete cracking was assumed in that analysis, and these additional wall deformations would generate additional forces in wall elements. These vicious cycles could lead to wall collapse if the original assumed wall thickness is inadequate because insufficient wall thickness leads to insufficient wall stiffness and that may cause wall collapses due to the P-Delta effect. Neglecting the Code required P-Delta analysis for the AP 1000 shield building wall is a mistake and could have a significant adverse effect on the safety of the shield building.

II.1.5 AP 1000 Shield Building Wall Does Not Meet NRC's Seismic Margin Requirement Because Actual In-Plane Shear Force (Demand) Significantly Exceeds Actual Ultimate In-Plane Shear Strength (Capacity)

Preventing core damage and controlling radiation release are important in nuclear power plant design. For new power plant structures that their failure may lead to core damage and radiation release due to containment failure, the NRC requires that a measure of seismic margin be established and demonstrated. The seismic margin of a structure is the difference between the structure's capacity (ultimate strength) and its strength at the seismic design basis (SSE), or between failure and functional. The failure or collapse of the shield building wall will cause the passive containment cooling water storage tank (PCCWST) and water inside it, which has a significant amount of weight, to crash and damage the steel containment and thus creates a potential core damage and uncontrolled release of radiation. Therefore, the design for the AP 1000 shield building is required to perform a Probabilistic Risk Assessment (PRA) based on seismic margins analysis (SMA) to ensure that the shield building possess the required seismic margin against seismic events. This seismic margin analysis (SMA) should demonstrate that the shield building structure possess a High Confidence (95%) of a Low Probability (5%) of Failure (HCLPF) against a Review Level Earthquake (RLE), which is two thirds greater than the seismic design basis earthquake or SSE (or 1.67 x SSE). The selection of a high 95% confidence in conjunction with a low 5% probability of failure is a conservative criterion for margins. The selection of the factor of 2/3 greater than the SSE for the RLE is another conservative criterion for margins. For a structure to achieve the HCLPF at RLE, it would have to be designed to a high structural strength or capacity, and thus the structure is extremely unlikely to fail during earthquakes. The NRC staff guidance for performing the seismic margin analysis is documented in reference 5.

The seismic design basis for the AP 1000 shield building is called Certified Seismic Design Response Spectra (CSDRS), which is the mathematical expression of the seismic design basis ground motion. The CSDRS for the AP 1000 shield building is equivalent to the more familiar term "Safe Shutdown Earthquake" (SSE) for individual nuclear power plants. AP 1000 shield building is designed to be functional in elastic state at the seismic design basis, or CSDRS, or SSE. For discussion purpose below, the more familiar term SSE will be used instead of CSDRS.

When the actual ultimate in-plane shear strength is exceeded by the actual applied in-plane shear force in a concrete element, the element fails by shearing.

The blue line in the drawing below represents the "capacity" of the in-plane shear strength of the wall specified by the ACI Code. The word "capacity" is also defined in the ACI Code as "ultimate strength," meaning that that value is the maximum or limiting for the shear strength of an element. The calculated in-plane shear force (stress) along the height of the shield building wall during the safe shutdown earthquake (SSE) and the review level earthquake (RLE) are also shown in the drawing. These two plots do not include the in-plane shear forces due to irregularity and improper modeling, accidental torsion, and P-Delta effects, as described above, because no analysis was performed by Westinghouse, and therefore they represent the magnitude of in-plane shear force less than the actual in-plane shear force in wall elements.

Notice that the in-plane shear force (red curve) due to the RLE [

] [a,c] Had the unaccounted for in-plane shear forces, such as that due to irregularity and improper modeling, accidental torsion, and P-Delta effects, been properly calculated and combined and added to the calculated in-plane shear force, and the ultimate in-plane shear strength properly reduced by the presence of out-of-plane shear force, as described above, the actual in-plane shear force during the RLE would have significantly exceeded the actual ultimate in-plane shear strength for the entire wall height.

Therefore, when the actual in-plane shear force [

] [a,c] it demonstrates that the AP 1000 shield building is far from being meeting the HCLPF requirement, or conversely, the corresponding earthquake ground motion level for the certified AP 1000 shield building's HCLPF point would be significantly lower than the RLE level. This is clear evidence that **the AP 1000 shield building does not meet the NRC's seismic margin requirement.**

Therefore, the in-plane shear strength of the concrete wall needs to be increased in order to meet the NRC's seismic margin requirement, and the GDC 2 requirement as described in the next section. Thickening the concrete wall is an effective, and probably the best, way to increase the in-plane shear strength for the AP 1000 shield building. By increasing the wall thickness, it also helps to resolve the inadequate punching shear strength problem of the shield building wall that would fail the NRC's aircraft missile impact requirement as will be described later.

a,c

II.1.6 No Adequate Demonstration that AP 1000 shield Building Meets GDC 2 Requirement

10 CFR Part 50, Appendix A, General Design Criterion (GDC) 2 requires, in part, that structures, systems, and components (SSCs) important to safety be designed to withstand the effects of earthquakes without loss of capability to perform their safety functions. The standards, which are used to ensure the safety function for a seismic Category I concrete structure, such as the AP 1000 shield building, are ACI 318 and 349 Codes, which are stated in the NRC Standard Review Plan, and endorsed in Regulatory Guide 1.142, "Safety-Related

concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments). ACI Codes require the safety function of a structure to be demonstrated through "strength", "ductility", and "energy dissipation/absorption capability." For strength demonstration, the criterion is that the concrete element/structure should possess strength (capacity) greater than the force (demand) that is applied to it. The calculated in-plane shear force (demand) during the SSE has reached [ ] a,c of the ultimate in-plane shear strength (capacity) specified by the ACI Code. However, this calculated in-plane shear force is less than the actual shear force because it excluded the shear forces due to the improper modeling, the accidental torsion analysis, and the P-Delta analysis, as described above. Therefore, there is no demonstration that the shield building meet GDC 2 requirement, and that demonstration is required. Furthermore, the shield building wall is a moderately tall wall and is the sole lateral support system for the entire shield building that supports a heavy water tank on top of the building roof. Therefore, a prudent design or review should certainly consider the recommended maximum in-plane shear strength of $\mathbf{6\sqrt{f_c'}}$, as stated in Section II.1.2, for design. If this prudent design consideration is adopted, the current insufficiently calculated in-plane shear forces have already exceeded the recommended in-plane shear strength – an indication that the shield building wall is not strong enough, and does not meet GDC 2 requirement.

II.1.7 NRC's Conclusion on the Safety of the Shield Building Subjected to Aircraft Missiles Contradicted with Structural Engineering Principles and Analysis Data

The aircraft missile analysis for the ESBWR DCD was performed by a reputable outside structural engineering firm that is specialized in this type of analyses, whose computer codes have produced analysis results that were substantiated by laboratory test, or field, data for decades. The analysis meticulously modeled the airplane and its impact to the reactor building wall, and the analysis results showed that a significant amount of airplane wreckage would penetrate through the ESBWR reactor building wall, which is 42 inch thick of ordinary reinforced concrete (see reference 6), and the wreckage caused cracking to the steel reactor head and its material stressed into plasticity.

However, Westinghouse performed its aircraft missile impact on the shield building in-house. Westinghouse has been an NSSS vendor, and not a structural engineering consulting firm, and its staff and computer codes have not been known to be able to conduct aircraft impact analysis for structures. NRC concluded that the aircraft missile would not penetrate through the AP 1000 shield building wall contradicted not only with that of ESBWR aircraft missile impact analysis but also with concrete structural engineering principles and theories. These principles and theories are described below to demonstrate the inadequacy of the NRC conclusion.

While the wall elements of the shield building would experience both in-plane and out-of-plane shear forces (stresses) simultaneously during earthquakes, only out-of-plane (radial) shear strength or punching shear strength is primarily involved in resisting the out-of-plane shear force generated by missile impact. Missile penetration occurs when punching shear strength of the concrete wall is less than the punching shear force. Punching shear strength is the summation of the out-of-plane shear strength (as indicated in the shear strength vs. ductility ratio diagram in Section I.2) along the periphery (perimeter) area near the punching object. When a missile strikes a building wall, the wall area in direct contact with the striking target will deform as much as the shear ductility capability of elements allows them to deform. If concrete elements possess excellent shear ductility capability, as the reinforced concrete element did in the test for a ductility ratio of four, as shown in the diagram in Section I.2, then the wall area in direct contact with the striking target will keep deforming in a dish shape and thus keep increasing the diameter or periphery of the dish shape of the wall. The larger of the shear ductility capability of concrete elements, the larger of the diameter of the dish shape, the more periphery (perimeter) area would be created, and thus the more punching shear strength for the concrete wall.

However, the shear strength of the AP1000 shield building element is less than that of the comparable ordinary reinforce concrete element, and has much less shear ductility, and significantly less shear energy absorption capability than that of the ordinary reinforced concrete element, as shown in the diagram in Section I.2. Therefore, the punching shear strength of the AP 1000 shield building wall is much less than that of the ordinary reinforced concrete building wall, such as the ESBWR's reactor building wall.

Therefore, It is not logical for the NRC to conclude that the 36 inch thick AP 1000 shield building wall would not be penetrated through by aircraft missiles, while the 42 inch thick ESBWR ordinary reinforced concrete wall would, which is stronger, and has more shear ductility, and significantly more shear energy absorption capability than that of the AP 1000 shield building wall,.

Therefore, the shear strength, shear ductility, and shear energy absorption capability of the AP 1000 shield building need to be upgraded to resist the punching shear force and energy generated by the aircraft missile impact. Thickening the concrete shield wall is an effective way to increase the punching shear strength. The increase of wall thickness also help resolve the inadequate in-plane shear strength problem as described in Section II.1.5, and II.1.6. Using ductile steel tie-bars (see discussion later), and reducing tie-bar spacing are the effective ways to increase the shear ductility and shear energy absorption capability that would be needed to resolve the inadequate punching shear strength and shattering problems, and meet the NRC aircraft missile impact requirement.

In addition to the missile penetration problem through the shield building wall, the PCCWST is likely to collapse through the shield building roof if it is struck by aircraft missiles because the support to PCCWST at the roof top of the AP 1000 shield building is flexible and does not appear to possess sufficient rotational stiffness, strength, and ductility to survive the missile impact force and energy as indicated in the author's e-mails, dated May 24, 2011 and June 11, 2011, both are attached. The May 24, 2011 e-mail is in the attachment to the June 11, 2011 e-mail. As stated in the May 24, 2011 e-mail, the other technical staff member, who was in charge of the AIA program, also raised concerns on the safety issue of the shield building subjected to aircraft missiles. However, the NRC reviewer ignored the request to correctly resolve the aircraft impact issue. Furthermore, the NRC reviewer who evaluated the aircraft missile impact for AP 1000 shield building had under-calculated missile impact force by a factor of [ ] [a,c] in another case (see attached e-mail from Dr. Chuang to the author, dated January 3, 2012), and therefore it is possible that he could have used the same method to calculate the aircraft impact force for the AP 1000 shield building.

The collapse of the PCCWST through the shield building roof would most likely crash and damage the steel containment due to the heavy weight of the PCCWST (including water weight), and create potential core damage and uncontrolled release of radiation. Although the collapse of the PCCWST is due to aircraft missile, not seismic, the end effect of core damage and radiation release is the same as that of shield building wall collapse due to seismic. Therefore the roof and the connections that supports the PCCWST should possess the same safety margin as specified in the NRC seismic margin requirement. However, the NRC did not address the concerns raised by its own structural engineers on the potential collapse of the PCCWST when it is struck by aircraft missile impact.

The shield building roof that supports, and connects to, the PCCWST, need to be redesigned and reanalyzed for its adequacy with the same safety margin as required by the NRC's seismic margin requirement.

II.2 Ductility

II.2.1 ACI Code Requires Ductile Design

Concrete is a brittle material, and therefore it needs reinforcement to make the combined reinforced concrete element behaving as a ductile element and acting as a unit. ACI 318-08 Code requires that each cross-section of an element/structure to possess a minimum flexural ductility ratio of 2.5 for ordinary sections that have low stresses and 3.75 for sections that have high stresses, which may not behave elastically under loads, and that the element/structure to eventually fail in ductile modes, such as flexure (bending), not in brittle modes, such as shear or torsion, to deform cyclically and provide high energy dissipation/absorption capability to concrete elements and the structure in order to survive earthquake ground motion and missile impact.

II.2.2 Building Collapse due to Insufficient Ductility and Code Revised Accordingly

The bombing of the Murrah Federal Office Building in Oklahoma City, Oklahoma resulted in collapse of a large portion of the building. As a result of the collapse, ACI Building Code added requirements for continuity of reinforcement. The intent was stated in ACI 318-08 Building Code, R7.13, to "improve the **ductility** and **redundancy** of structures so that in the event of damage to a major supporting element or an abnormal loading event, the resulting damage may be confined to a relatively small area and the structure will have a better chance to maintain overall stability." Field data from impact loading or earthquakes, the laboratory testing data, and analytical research, have all demonstrated that "ductility" is the most important and desirable characteristic of an element/structure to resist seismic and impact loadings.

Redundancy is another characteristic that prevent structures from collapse by providing an alternative load path in case the original designed load path failed. Redundancy is inherited in the frame-type of buildings, such as the Murrah building, because floor slabs not only tie columns and walls together but also redistribute the vertical weight or lateral force in two orthogonal directions to supporting framing systems. Therefore, if one load path is damaged or lost, the other load path can pick the load up, thus avoiding building collapse. The Murrah building did possess ductility and redundancy, but the collapse of a large portion of the building was due to insufficient ductility and redundancy and not with no ductility or no redundancy.

II.2.3 AP 1000 Shield Building Has No Redundancy and No Ductility in Radial Direction

The cylindrical shell type of structures, such as the AP1000 shield building, has no redundancy, because it does not have floor slabs to tie columns and walls together and cannot redistribute the vertical weight and lateral force through floor slabs like the frame-type of structures, such as the Murrah building did. With no redundancy, the AP1000 shield building wall is limited with punching shear strength, shear ductility, and shear energy absorption cability to resist the impact force from missiles. When the wall is struck by an aircraft missile, the wall will deform as much as its shear ductility allows to bring a larger area than the missile target area to resist the impact force and energy. If the element is brittle in shear, as the test data shown, for the AP1000 shield building, only the punching shear strength of the element near the missile target area can be mobilized to resist the impact force, which is small because the area that can deform is small. Not only the element being struck will be punched through due to the insufficient punching shear strength of a small area, the region near the missile target area could collapse due to the shattering effect, as stated by the impact testing laboratory in Section I.1, and as occurred in the Murrah building to certain extent. Low ductility, low redundancy, and low toughness (energy dissipation capability) were attributed as factors in the cylindrical water tank structure collapse in reference 7. **No redundancy, no ductility in the radial direction (out-of-plane shear), and the shattering effect of** the certified AP 1000 shield building subjected to seismic or impact forces should be a frightening condition to any knowledgeable and experienced structural engineers.

II.2.4 NRC's Decision and Action to Accept the Brittle Concrete Element is improper, Against Structural Engineering Principles, and Violets ACI Code's Requirements

When a properly specified and executed test fails, it is the demonstration that the design was inadequate and a redesign is needed until the new design meets the acceptance criteria. This approach meets structural engineering principles and is required by the ACI Code and has been practiced by the structural engineering profession with success for a long time. However, after the concrete element failed in a brittle manner and far from being meeting the acceptance criteria, the NRC accepted the brittle concrete element by ignoring the approach and acceptance criteria for the design of concrete elements, which were established with the help of the two NRC consultants, who are the world leading experts in seismic design for concrete shell types of structures, and through several days of meetings with, and accepted by, Westinghouse. Therefore, the NRC's decision and action to reverse its carefully established testing approach and acceptance criteria to achieve the required ductility for the concrete element and accept the brittle concrete element is improper because accepting brittle concrete element is against structural engineering principles that requires ductile concrete elements, and violets the ACI Code's requirement that prohibits the use of brittle concrete elements.

II.2.5 NRC's Justification for Accepting the brittle concrete Element Is Wrong

The NRC's justification for accepting the brittle concrete element is "The Code does not specify a ductility level nor does it specify that ductility should be in every single structural component of the structure" (see reference 8). This justification is seemingly appearing because the ACI Code does not use English language to specify ductility and ductility level as stated in the quotation by the NRC in its justification. However, the Code requires all sections (elements) in structures to be designed in such a way that the concrete would not be crashed before the steel reach a minimum strain value of 0.005 in./in. for sections that have low stresses, and 0.0075 in./in. for sections that may have high stresses. These numerical steel strain values correspond to a ductility ratio of 2.5 for sections that have low stresses and 3.75 for sections that have high stresses, based on the theory or method of concrete design, as derived and calculated by the author in reference 8, which is attached. Therefore, the NRC's justification for accepting the brittle concrete element was resulted from misinterpreting the ACI Code, and is wrong.

II.2.6 NRC's Justification for the Safe Design of the Shield Building by Accepting the Nonlinear Static Pushover Analysis and Its Results Is Without Technical Merit and Wrong

The NRC's justification for the safety of the AP 1000 shield building is, "The push-over method is an accepted industry practice for estimating the limit state (i.e., collapse load) and corresponding modes(s) of failure of a structure due to seismic loading…." (See reference 8). Had the use of the pushover analysis method been a viable way to justify the safety for the AP 1000 shield building, the NRC would not have had the need to hire the two consultants to help it to establish the acceptance approach and criteria for the design of new concrete elements. Furthermore, If the pushover analysis method were able to justify the safety of the seismic Category I AP 1000 shield building in place of the ACI Code's procedures, criteria, and requirements, there would be no need for the existence of the ACI Code, and for structural engineers to possess sufficient knowledge and experience in structural analysis and design methodologies. By applying the NRC's logic, any person can use, and follow the instructions of, the nonlinear static push-over computer code, to design and review structures. However, that is not structural engineering. Safety review of structures require knowledgeable and experienced structural engineers, who are educated and understand structural engineering principles and go through ACI Code's rigorous prescribed procedures, criteria, and requirements to confirm that the analysis and design of the structure does not violets structural engineering principles and is in compliance with the Code's procedures, criteria, and requirements. The NRC reviewer must possess sufficient structural knowledge and experience to identify the misuse of analysis and design methods submitted by applicants.

The nonlinear <u>static</u> pushover analysis method for the shield building uses a concentrated horizontal force applied at the top of the shield building wall and keep pushing it <u>statically</u> in one direction until materials in the wall yields (thus the name of nonlinear in front of the push-over analysis method) and eventually failed in either concrete crushing in the compression side of the wall or steel rupture in tension side of the wall. This type of analysis assumes that the failure mode is flexural or bending, but not shear and/or torsion. Although the flexural failure mode of the shield building needs to be investigated among all possible failure modes, it is wrong for the NRC to conclude the safety of the shield building solely based on this failure mode. In fact, the most likely failure mode, or the controlling failure mode, for the AP 1000 shield building during earthquakes, is shear with combination of torsion due to the highly irregular shapes of boundary (supported by the zigzag ordinary reinforced concrete structure below and supporting the auxiliary building roof at different locations of the shield building) conditions of the shield building that is formed by the brittle concrete elements. The NRC failed to discern that Westinghouse had only picked the wrong failure mode for the analysis by using the static pushover analysis method, among all possible failure modes. Had the NRC followed its SRP guidance and the NRC endorsed ACI Code procedures, criteria, and requirements, it would have found that other types of failure modes would render the shield building design inadequate. Furthermore, the NRC failed to understand that the nonlinear <u>static</u> pushover analysis is not applicable to the irregular, tall, and safety important AP 1000 shield building even for the flexural failure mode, and this inapplicability is discussed below.

With respect to the applicability of the pushover analysis method, the NRC staff states "The push-over method is an accepted industry practice for estimating the limit state (i.e., collapse load) and corresponding modes of failure of a structure due to seismic loading….." The fact is that the static pushover analysis neither is an accepted industry practice nor is the method can predict corresponding modes of failure of a structure due to dynamic loads from earthquake ground motion because the structure moves cyclically, but not in one direction as the push-over analysis assumed. The static pushover analysis method can only produce one failure mode that is due to flexure (bending). However, a structure vibrates in many modes and mode shapes during earthquakes and requires dynamic analysis to capture the dynamic behaviors and failure modes, such as shear, torsion, buckling, and the combinations of them.

On the applicability of the pushover analysis method, the National Earthquake Hazards Reduction Program (NEHRP), 2009 edition, states, " **Applicability**. **Regular structures less than 40 feet in height in Occupancy Categories I and II** maybe designed using the nonlinear static procedure following the requirements of this chapter," and the reason for these limitations of applicability is stated "**FEMA 440 identifies significant disparities between response quantities determined by nonlinear static analysis and those determined by nonlinear dynamic analysis for all but low-rise structures; therefore, use of the nonlinear static procedure for the design of members proposed here is limited to structures 40 feet or less in height.**" The relevant statements of the NEHRP are highlighted and attached. The last page in the attachment indicate that occupancy categories I and II are for ordinary structures, occupancy category III is for high occupancy structure, and occupancy category IV is for essential structures. The AP 1000 shield building is an essential structure. The nonlinear static push-over computer analysis method is a static analysis, and naturally cannot capture the dynamic behaviors of structures subjected to earthquakes, which are dynamic loadings that require dynamic analysis. Therefore, the static pushover analysis method maybe used only for small regular shapes of structures that are neither with a lot of people in it nor is its failure will cause detrimental effects. The "**Regular structures less than 40 feet in height**" is important with respect to the applicability of the pushover analysis method, because the behavior of regular shape of low-rise structures during earthquakes may be roughly predicted by the static pushover analysis method.

However, the shield building is a highly irregular structure due to its boundary conditions and with a heavy water tank on top of its roof, has a height of about 270 feet, and should be classified as occupancy category IV. Therefore, the nonlinear static pushover analysis is clearly not applicable to the AP1000 shield building. More detail reasons for the inapplicability of the method to the shield building were described in reference 8, which is attached. Therefore, the NRC's justification for the safe design of the AP 1000 shield building by solely using the nonlinear static pushover analysis is without technical merit, and wrong.

The two technical justifications that underpinned the NRC's decision and action to accept the brittle concrete element for, and the safe design of, the shield building, have been proven to be incorrect, baseless, and without technical merit. With so many analysis and design deficiencies as described above, **the certified AP 1000 shield building, which is weak and brittle, will**

become sitting nuclear bombs, at different locations in the world, waiting for explosion, when they are triggered by aircraft missiles or earthquakes, because its design violets structural engineering principles, and does not meet the NRC's seismic margin requirement and aircraft missile impact requirement.

The NRC staff has not addressed the **shattering effect** as a result of impact loading on the brittle element/structure of the shield building when it is struck by missiles, a phenomenon of a brittle element/structure, as stated above by the testing laboratory, and as had occurred to certain extent for the less ductile Murrah building.

II.2.7 The Importance of Shear Ductility by Providing Steel tie-bar Reinforcement

Concrete cracks when the applied tensile or shear force exceeds its corresponding strength of concrete elements, and cracks easily propagate through brittle concrete elements to larger areas due to the lack of steel tie-bar reinforcement to provide ductility and arrest concrete cracking. The AP 1000 shield building wall has the same cylindrical shape as that of Crystal River nuclear power plant containment. Due to force interaction effects that was not accounted for in the design, and the lack of steel tie-bar reinforcement in the region, was determined to be the root cause of the partial collapse of the Kaiga nuclear power containment dome in India, and of concrete delamination in the dome and wall of the Crystal River plant. They are all repaired by adding steel tie-bar (stirrup) reinforcement in the radial direction (perpendicular to the thickness direction of the wall), except the repair work for the Crystal River containment wall is ongoing. These tie-bars are similar to the tie-bars in the concrete element of the AP1000 shield building. The steel tie-bar reinforcement not only increases the shear strength of concrete elements but more importantly increases shear ductility of the concrete element. **ACI Code requires minimum steel tie-bar (stirrup) reinforcement to provide shear ductility to concrete elements even when the calculated shear force is less than the Code specified ultimate shear strength.**

The force interaction effects are not mentioned in the ASME, Section 3, Division 2, "Code for Concrete Containments. " The NRC staff has requested the ASME, Section 3, Division 2, Code Committee to add design requirements to address force interaction effects and add radial steel tie-bar (stirrup) reinforcement for the containment design. The staff also requested the MHI to address the force interaction effects in its US-APWR concrete containment design. The above failure lessons due to force interaction effects and the lack of ductility in concrete elements without steel tie-bar reinforcement in the region, the repair method to provide ductility to concrete elements by adding steel tie-bar reinforcement, the ACI Code's ductility concept and requirement for providing steel tie-bar (stirrup) reinforcement for the shear ductility reason, and the staff actions to the ASME Code Committee, should be a good proof and confirmation to the NRC on the importance and necessity of the NRC staff's original acceptance criteria for the AP 1000 shield building, that is, a concrete element should possess a ductility ratio of three (3) for both in-plane and out-of-plane shears. The above discussion should also provide the NRC with enough evidence and warning to reverse its wrong decision and action in accepting the brittle concrete element, due to insufficient steel tie-bar reinforcement, to make the concrete element ductile so that the shield building can meet the NRC's seismic margin and aircraft missile impact requirements.

II.3 Energy Dissipation/Absorption Capability

II.3.1 Energy Dissipation Capability for Earthquake

For earthquake-resistant buildings, ACI 318-08 Code, Chapter 21, Earthquake-Resistant Structures, Section R21.1.1, states,

> "The design and detailing requirements should be compatible with the level of energy dissipation (or toughness) assumed in the computation of the design earthquake forces. The terms "ordinary," "intermediate," and "special" are specifically used to facilitate this compatibility. The degree of required toughness and, therefore, the level of required detailing, increases for structures progressing from ordinary through intermediate to special categories. It is essential that structures assigned to higher SDCs (seismic design category) possess a higher degree of toughness"

The AP 1000 shield building belongs to the higher seismic design and "special" category, and requires the higher degree of toughness (energy dissipation capability) in accordance with the ACI Code. Therefore, the design for the AP 1000 shield building requires it to achieve a very ductile concrete element/structure to provide excellence energy dissipation/absorption capability to survive earthquakes. For ordinary and intermediate structures, ACI Code does not require special ductility control for steel reinforcing bars. However, for special category moment frames or structural walls, to make the design of ductile concrete elements to provide excellent energy dissipation/absorption capability possible, ACI Code requires special ductility control for reinforcing bars as follows:

> **21.1.5 — Reinforcement in special moment frames and special structural walls**
>
> **21.1.5.1** — Requirements of 21.1.5 apply to special moment frames, special structural walls, and all components of special structural walls including coupling beams and wall piers.
>
> **21.1.5.2** — Deformed reinforcement resisting earthquake-induced flexure, axial force, or both, shall comply with ASTM A706, Grade 60. ASTM A615 Grades 40 and 60 reinforcement shall be permitted if:
>
> (a) The actual yield strength based on mill tests does not exceed f_y by more than 18,000 psi; and
>
> (b) The ratio of the actual tensile strength to the actual yield strength is not less than 1.25.

The actual yield strength for steel is limited to 78,000 psi in accordance with 21.1.5.2 (a), and the reserve between the tensile strength and yield strength must be not less than 25% in accordance with 21.1.5.2 (b). ASTM A706 steel is about twice as ductile as ASTM A615 steel, and therefore it is the preferred steel for special frames and structural walls, as specified. The Code permits the use of A615 steel provided it meets the two ductility requirements in 21.1.5.2 (a) and (b) because the two requirements provide ductility and energy dissipation/absorption capability to the concrete elements and the structure. As can be seen in the attached file, among the four batches of tests for A615 steel, only steel of batch (a) passes the Code's requirements, and the steel in other three batches failed the Code's ductility requirements both (a) and (b). The second page of the file is the actual stress-strain curves for the tie-bar steel of the AP 1000 shield building. The steel reached about [ ] a,c at the strain of [ ] a,c and became unstable and reversed downwards while the A615 batch (a) steel yielded at about [ ] a,c and kept going upward steadily to [ ] a,c and then flatten into plasticity but still stable. Therefore, the tie-bar steel for AP 1000 shield building not only does not meet the Code's requirements but was behaving in the opposite direction to the Code's ductility requirements. Although the Code Section 21.1.5 is supposed to apply to main reinforcement, and not the tie-bar steel, the brittle tie-bar steel certainly does not meet the spirit of the Code and is not a good choice for the design of a ductile concrete element. This is one of the reasons why the concrete element failed in a violent brittle manner during the out-of-plane shear test.

It takes ductile steel material and a ductile design to make a ductile concrete element, and only with the ductile concrete element that can form the ductile cylindrical shell shield building wall that possesses energy dissipation/absorption capability to be the seismic Category 1 earthquake-resistant structure to resist the imparted energy from earthquake ground motion.

II.3.2 Energy Absorption Capability for Missiles

For impact-resistant buildings, ACI 349-06, "Code for Requirements for Nuclear Safety-related Concrete Structures and commentary," section RF.4-requirements to assure ductility states,

> "The provision to assure ductility are parallel to appropriate sections of Chapter 21 of ACI 318-05."

Ductility and energy absorption/dissipation capability are interchangeable because the more ductile the element/structure is the more energy absorption/dissipation capability it possesses."

From the drawing of shear strength vs. ductility ratio curves in Section I.2, it is obvious that the brittle concrete element/structure of the AP1000 shield building **does not meet the ACI Code requirement with respect to the energy dissipation/absorption capability for missiles** because the triangular area in red color, which is the energy absorption capability of the AP 1000 shield building element, is very small. Therefore, the NRC's conclusion that aircraft missiles would not penetrate through the shield building wall contradicted with the principles of energy absorption capability of concrete in the ACI Code, and the Westinghouse test data on the brittle wall element.

III. Proper vs. Improper Ways of Regulatory Review

The author has illustrated a review process, base on structural engineering principles, and the NRC endorsed ACI Code's procedures, criteria, and requirements, which is a proper way that has been established by the structural engineering profession and practiced daily by structural engineers in their design or review of structures. That proper way of review demonstrates that the certified AP 1000 shield building does not meet the NRC's seismic margin requirement and the aircraft missile impact requirement, and thus is unsafe. On the contrary, after the test concrete element failed in brittle manner and was far from meeting the acceptance criteria, the NRC simply made a decision to accept the brittle concrete element by ignoring the acceptance approach and criteria for the design of wall elements that it established, with the help of two world leading authorities in concrete seismic design, in accordance with the ACI Code's testing requirement, and agreed by Westinghouse, with an incorrect justification by wrongly stating that ACI Code neither requires every element to be ductile nor specify a specific ductility value for elements while the Code does require both. Furthermore, in addition to accept the brittle concrete wall element, the NRC concluded the shield building design was safe, by accepting Westinghouse's nonlinear <u>static</u> pushover analysis results with the justification that the pushover analysis method is an accepted industry practice while the method is not applicable to the <u>seismic (dynamic)</u> analysis for the safety important AP 1000 shield building. Clearly, the NRC's arbitrary decision, action, and justifications are, without technical merits, illogical, improper, and unprecedented in the modern history of structural engineering known to the author.

IV. Consequence of Decisions

IV.1 Bad Decisions Resulted in Collapse and Explosion

The walkway collapse in the Kansas City Hyatt Regency, killing 114 and injuring more than 200 people, was caused by a structural engineer's bad decision to delegate his design responsibility to a steel fabricator for connection design without checking whether the fabricator's design is adequate or not due to time or schedule constraints. That bad decision resulted in that **the design force (demand) in the steel connection rod exceeded the strength (capacity) of the rod, similar to the situation of the AP 1000 shield building design that the actual in-plane shear force (demand) exceeded the actual ultimate in-plane shear strength (capacity)**. The tragedies of shuttle Challenger and shuttle Columbia were mainly caused by bad decisions of pushing schedules over resolving known safety issues. In the walkway collapse case, the structural engineer did not know that the design force (demand) had exceeded the design strength (capacity) of the connection due to his negligent. In the challenger case, the engineer worried that the O-ring might not work due to the cold weather and recommended to delay the launch, but was over ruled by the management due to scheduling consideration. In the Columbia case, NASA had the small insulating form broke-off incidents in prior trips without accidents. However, the NRC made a decision to accept the AP 1000 shield building knowing that the concrete element failed in brittle manner and was far from being meeting the acceptance criteria that it established jointly with Westinghouse, but chose to ignore the fact, and went even further to stop the review process and released Westinghouse from performing ACI Code required analyses that are necessary for safety evaluations of the building. This decision and action is unprecedented, illogical, and inappropriate. The tragedies in the Kansas City Hyatt Regency, shuttle Challenger, and shuttle Columbia, were each caused by one technical deficient design: connection in the Kansas City Hyatt Regency, O-ring in the Challenger, and the insulating form in the Columbia. As described above, the AP 1000 shield building has numerous design deficiencies: improper mathematical modeling, incomplete analyses, substituting static pushover analysis for required seismic (dynamic) analysis, and for replacing ACI Code's procedures, criteria, and requirements, insufficient strength and ductility of wall concrete elements. These numerous known design deficiencies should be eliminated.

VI.2 Good Decision Protects Public Safety and Health

During the October 2011 ASCE annual conference, participants at the ASCE ethics seminar noted that while many clients can be demanding, the engineer has a greater obligation to ensure public safety, health, and welfare. The discussion of this ethics issue is documented in "Standing Up and Pushing Back," the Magazine of ASCE, January 2012, which is attached.

The case is about a civil engineer, who was assigned to collect soil samples and perform percolation tests for a new apartment complex development project. Two out of four tests failed the percolation rate. If the soil does not percolate sufficiently, runoff could contaminate the Barton Spring Creek in Austin, Texas. The engineer requested additional tests due to the 50% of failure rate, but was rejected by his manager. An excerpt from the paper states "The manager insists that the firm has never missed its dead-lines and that the developer is not a client she wishes to alienate. She states that the engineer needs to "make it work" with the data he has and that if a few results need to be scrapped because of possible errors, then that is what he should do."

The manager's desire was to serve her client, but the engineer considered his duty to protect the public interest should be above the financial interest of the client, and the desire of his manager. Because of the integrity, professionalism, and courage of the engineer, he protected the quality of water of the Barton Spring Creek for the safety and health of the public.

V. Conclusion

A structure is safe if it is properly analyzed and designed. A structural analysis is adequate if a proper analysis method is used. A structural design is adequate if a proven design method is used. A design method is proven to be adequate if it was derived from, or substantiated by, tests.

The AP 1000 shield building uses a new type of concrete element that the ACI Code's design equations do not apply, and is required by the Code to demonstrate the adequacy of the design for the new concrete element by testing. The NRC staff, assisted by two world leading authorities in seismic design, met with Westinghouse for several days and jointly established acceptance criteria for the design of the new concrete element and a test program to verify the design adequacy of the new concrete element. The design process for the wall is (1) to verify that the design of the new concrete element is sufficiently strong and ductile as building blocks for the shield building wall by meeting the acceptance criteria through testing, and (2), the wall would then be analyzed and designed in accordance with ACI Code's procedures, criteria, and requirements.

One concrete element, with a design of widely spaced steel reinforcement, failed in a brittle manner during the out-of-plane shear test, and another element, which was incorrectly built, and improperly tested for in-plane shear. Both concrete elements were far from being meeting the acceptance criteria.

After the failure of the tests, Westinghouse submitted analysis results by a nonlinear static pushover analysis computer code to replace the agreed upon testing program and acceptance criteria for the design of the new concrete element and the ACI Code's analysis and design procedures, criteria, and requirements for the entire shield building wall. While the two experts and the author insisted that the brittle concrete element should not be accepted and the new concrete element should be redesigned and retested, the NRC made a decision to accept the brittle concrete element and the shield building as being safe. The NRC's justification for accepting the brittle concrete element is "The Code does not specify a ductility level nor does it specify that ductility should be in every single structural component of the structure," which is incorrect because the Code does require both and the author had demonstrated that. The NRC's justification for accepting the static pushover analysis method and results is "The push-over method is an accepted industry practice for estimating the limit state (i.e., collapse load) and corresponding modes of failure of a structure due to seismic loading..," which is also incorrect because building Codes and the National Earthquake Hazards Reduction Program (NEHRP) prohibit the use of static pushover analysis for seismic (dynamic) analysis for irregular, or tall, or safety important structures, while the AP 1000 shield building is an irregular, tall, and safety important structure.

The NRC's decision to accept the AP 1000 shield building design not only short circuited the review process that was ongoing and incomplete but also released Westinghouse from completing necessary ACI Code's required analyses, and without those analyses results, a safety conclusion is difficult to assess.

However, based on the procedures, criteria, and requirements of the ACI Code, the author has demonstrated that (1) the certified AP 1000 shield building does not meet the NRC's seismic margin requirement, (2) there is no adequate demonstration that the shield build meets the GDC 2 requirement, (3) the NRC's conclusion that the aircraft missile would not penetrate the AP 1000 shield building wall is illogical, and (4) the shield building wall is insufficiently strong and ductile to resist earthquakes or aircraft missiles. The collapse of the roof or wall will cause the heavy water tank to fall and crash the steel containment and potentially damage the cores and create uncontrolled radiation release. Therefore, in order to meet the GDC 2 requirement, the NRC seismic margin requirement, and the aircraft missile impact requirement, design modifications to the shield building

are required. The required modifications include the increase of the wall thickness and ductility, and a redesign of the connections between the roof and the water tank. Without design modifications to the certified AP 1000 shield building, which is weak and brittle, it will become sitting nuclear bombs, at different locations in the world, waiting for explosion, when they are triggered by aircraft missiles, or earthquakes.

The author hopes that his demonstration provided sufficient evidence that will cause the NRC to make a right and wise decision to require design modifications to the unsafe AP1000 certified shield building design and make it safe so that the safety, health, and welfare of the public can be protected.

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DUCTILITY IN SHEAR RESPONSE OF SHIELD BUILDING

Frank Vecchio
July 9, 2010

Code Requirements:

The intent of Chapter 21 of ACI-349 (Provisions for Seismic Design) is that a structure be designed to behave, at its ultimate limit state, in a ductile manner. This necessitates that the structure exhibit a failure mechanism that involves yielding of the principal reinforcement such that a high deformation capacity be achieved, providing sufficient energy dissipation and avoiding all brittle failure mechanisms. The Code does not provide specific guidance on the levels of ductility that are required.

A generally accepted criterion for assessing adequate ductile behavior does not exist amongst jurisdictions or design code worldwide, although many are working toward criteria based on the concept of displacement or ductility demand. The criterion proposed in New Zealand, for example, states that if the structure can withstand four cycles at four times the yield displacement with no more than a 20% decay in force capacity, then it is adequately designed to resist high seismic loading. It is understood that this is a highly stringent criterion, particularly when isolating the behaviour in a single member or joint. Given the size and nature of the Shield Building (SB), three cycles at a displacement amplitude of three times the yield displacement with no more than a 20% reduction in strength would, in my opinion, be adequate evidence of good ductile behaviour.

Although ACI-349 does not define specific target levels for ductility, it is worth noting that it implicitly requires that the failure mechanism be ductile regardless of the magnitudes of the actual design loads. Thus, for shear design of flexural members, Clause 21.3.4.1 states that "…the design shear force shall be determined from consideration of the statical forces on the portion of the member between the faces of the joints. It shall be assumed that moments of opposite sign corresponding to the **probable flexural moment strength** act at the joint faces…". In other words, the design shear force is dictated by the flexural capacity of the member, ensuring that a ductile flexural failure occur before a brittle shear failure can develop. In the case of the SB wall, the ¾-inch steel faceplates create a large moment capacity, and thus the shear capacity required to maintain a ductile failure mechanism is high, regardless of the actual out-of-plane shear forces acting.

It is understood that the aforementioned requirements of Chapter 21 of ACI-349 relate primarily to moment-resisting frames, and not directly to shell structures such as the Shield Building. [

] a,c

Nevertheless, Chapter 21 is based on the important underlying principle that if, in the event of unforeseen circumstances, the loads acting on the structure are of much larger magnitude than anticipated, then the structure should be able to achieve a ductile failure mechanism. [

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Comments on the Safety of AP1000 Shield Building

By Thomas T. C. Hsu and Y.L. Mo

Nov. 14, 2010

1. Building Codes for Seismic Design

The requirements and recommendations in current seismic codes are based on the present knowledge about earthquakes and structural performance. Although many seismic provisions are available and worthy of discussion, such as the Eurocode 8 used by the European Community, the International Building Code is the prevalent "model code" in the United States, as explained in the following paragraph.

The International Building Code was first published in 2000 by the International Code Council (ICC), a US-based, non-profit, non-governmental, membership association. This code is based on the provisions contained in the NEHRP Recommended Provisions for Seismic Regulations for New Buildings, issued by the Building Seismic Safety Council (BSSC) of the National Institute of Building Sciences (NIBS), under the sponsorship of the Federal Emergency Management Agency (FEMA). This National Earthquake Hazard Reduction Program (NEHRP) publication is the outcome of a program initiated by FEMA to develop an up-to-date set of seismic provisions that could be adopted by building authorities all across the United States.

2. Equivalent Lateral Force method vs. Dynamic Method

According to the International Building Code, two methods for earthquake-resistant design are defined: 1) the Equivalent Lateral Force method, and 2) the Dynamic Method. The Equivalent Lateral Force method is applicable in any seismic zone to regular structures under 240 ft in height and to irregular structures of not more than five stories nor over 65 ft in height. The Dynamic Method may be used for any structure, but must be used for structures over 240 ft in height, irregular structures over 5 stories or 65 ft in height.

In the Dynamic Method, a modal response spectrum analysis is performed (Villaverde, 2009; Paz and Leigh, 2004). The International Building Code requires that the mode shapes, natural periods, and participation factors of the structure be determined. It also requires including as many modes as necessary to take into account the high modes and to obtain a combined participating mass of at least 90% of the actual building mass in each of the two orthogonal horizontal directions of the structure. When the base shear force calculated by the Dynamic Method is less than that determined by the Equivalent Lateral Force method, then for irregular buildings, the base shear calculated by the Dynamic Method shall be scaled up to match 100% of the base shear determined by the Equivalent Lateral Force method.

For the seismic design of the AP1000 shield building it is obvious that the Dynamic Method must be employed, because of its height and significant irregular boundary (support) conditions. In addition to the above code mandate from a structural point of view, it is only prudent to conduct a dynamic analysis on the AP1000 shield building, considering its stringent safety requirements, high construction cost, and shining public image.

3. Dynamic Method

At present, the Dynamic Method requires a modal response spectrum analysis. This analysis could help evaluate the strengths and deformations of the structures. Therefore, an acceptable design must satisfy the following principle: The strength, the ductility and the detailing must all be sufficient to resist the imposed earthquake action.

It must be acknowledged that the methodology of evaluating the supply and demand of a structure to resist earthquake still remain to be established. On the supply side, we are just beginning to understand the structural performance under cyclic loading, the nature of hysteretic loops, the cyclic ductility, and the energy dissipation. On the demand side, we are still unsure of the nature of the earthquake as to time, frequency of occurrence, intensity, location relative to the structure; earthquake energy; media of the source, propagation path, condition at the site, duration of shaking, frequency composition of the disturbing waves, and other factors. No other structural loadings involve such a wide variation of possible demands.

Until a more rational methodology is developed in the future to connect the supply and demand in seismic design, such as the Incremental Dynamic Analysis (IDA), the most sensible approach to design the nuclear containment structure is to recognize the uncertain nature of the earthquake demands, and to provide the ductility that could redistribute the stresses so that all the reserve capacity could be mobilized to avoid a catastrophic failure. Hence, the ductility and the proper detailing of the structures to prevent brittle failure at possible weak locations are crucial.

4. Brittle Failure of SC Module with [ ] [a,c]

The AP 1000 Shield Building is constructed of concrete wall with steel cover plates. The two cover plates of this steel-concrete (SC) wall is connected by tie bars serving as transverse shear reinforcement. The tie bars proposed by WEC are [ ] [a,c] in both the directions of height and circumference. However, tests at Purdue University showed that the SC modules failed by out-of-plane shear in a brittle manner. The question before us is: Is the SC wall with [ ] [a,c] safe?

WEC suggested that the wall with [ ] [a,c] is safe, because the out-of-plane shear stresses calculated by the pushover analysis is very small compared to the brittle shear strength of the wall. So the shear stress will never reach the shear strength to cause a brittle shear failure.

The writers acknowledge that the out-of-plane shear stress calculated by the static pushover analysis is indeed very small. However, the stress in the cylindrical wall [ ] a,c is actually under combined stresses of out-of-plane shear stress, in-plane shear stress, and axial stresses. [ ] a,c Furthermore, as stated previously in Section 1, the static pushover analysis, which is a type of Equivalent Lateral Force method, is not applicable to the shield building because of its height and significant irregular boundary (support) conditions.

More importantly, the requirement of ductile shear failure is not intended to guard against the loading assumed in the static pushover analysis. The ductility requirement is intended to guard against all the unexpected loadings that are not designed for. In the multitude of possible loadings, deformations and combinations, where the resulting shear stresses could exceed the shear strength, the wall could redistribute the stresses and could mobilize all the reserve capacity to avoid a catastrophic disaster. In the opinion of the writers, a ductility ratio of 3 required by NRC is a reasonable one and needs to be implemented. The NRC ductility requirement is actually more liberal than the ductility ratio of 4 required by the New Zealand Code.

5. Two Examples of Catastrophic Brittle Failures of Buildings

(A) Warehouses at Wilkins Air Force Depot in Shelby, Ohio, and Robins Air Force Base near Macon, Georgia.

At the Wilkins Air Force Depot in Shelby, Ohio, about 370 m^2 (4,000 ft^2) of the roof collapsed suddenly on August 17, 1955 (Feld 1964, p. 25). A similar warehouse roof collapse took place at Robins Air Force Base near Macon, Georgia, early on the morning of September 5, 1956.

The Ohio warehouse was a six-span rigid frame building, 122 m (400 ft) wide and 610 m (2,000 ft) long. The haunched rigid frames each had six 20 m (67 ft) spans, and were spaced approximately 10 m (33 ft) on center. In these warehouses, the girders have no stirrups and failed in a brittle manner, causing catastrophic collapse.

At the time of collapse, these two structures were subject to wider temperature variations. The temperature stresses, combined with shrinkage and shear effects to cause high tensile stress. The rapid, monolithic casting of the frames was thought to exacerbate shrinkage and to contribute to the problem (Feld and Carper 1997, pp. 255 – 257). Real structures do not behave in the same way as our simplified analytical models, and develop forces and stresses where our analyses suggest there should be none or very small.

These two warehouses illustrate the importance of designing beams with stirrups (tie-bars) that fail in a ductile manner. This lesson led to the establishment of a minimum shear reinforcing steel requirements in subsequent editions of the ACI Building Code.

(B) Mineo Manufacturing Plant in Miami, Florida

When the first writer was a professor at the University of Miami in the 1970s, he was asked to investigate the collapse of a roof structure in a manufacture plant of Mineo Company. The roof, which was made up of precast prestressed T-beams, collapsed during a stormy night. The precast prestressed T-beams, which were manufactured by a local prestressed concrete company, contained no stirrups (tie-bars). The reason this company were allowed to manufacture beams without stirrups (apparently in violation of the ACI minimum stirrup requirement) was because they tested a series of such beams (without stirrups) in their plant and found that they were able to carry the predicted ultimate loads. The beams were tested in a simply supported manner (i.e. no restraints to the beam movement along the length of the beam) and carried vertical loads only.

So why did the beams performed adequately at the test site, but collapsed catastrophically on the roof of the Mineo manufacturing plant? The answer is actually quite simple. It was because the beams at the Mineo manufacturing plant were subjected to a very different set of loadings than they were tested. In addition to the vertical loads, the beams were subjected to a longitudinal tension and a bending when the steel seating plates at both ends were welded to the support columns. Also, the beams were subjected to vibration when the roof was lifted up by the suction of the wind above the roof. The beams collapsed under these unexpected loadings that were not designed for.

A more important reason for the collapse was that the beams without stirrups failed in a brittle manner. The beams could not mobilize all the reserve capacities to resist the unexpected loadings. In other words, design based on strength alone is insufficient to ensure the safety of the structure. Ignoring ductility could lead to a catastrophic disaster.

It is interesting to point out that several roof collapses occurred in Miami following the collapse at the Mineo manufacturing plant. The beams involved in the subsequent collapses were all beams without stirrups (tie-bars). As a result of this series of roof collapses, the South Florida Building Code banned the use of beams without stirrups.

References

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Paz, M. and Leigh, W. (2004). Structural Dynamics Theory and Computation, Fifth edition, Springer.

Feld, J., and Carper, K. (1997). Construction Failure. 2nd Ed., John Wiley & Sons, New York, N. Y.

McKaig, T. (1962). Building Failures: Case Studies in Construction and Design. McGraw-Hill, New York, N. Y.

RESUME OF THOMAS T. C. HSU

Thomas T. C. Hsu is a John and Rebecca Moores Professor at the University of Houston (UH), Houston, Texas. He received his MS and Ph.D. degrees from Cornell University and joined the Portland Cement Association, Skokie, Illinois, as a structural engineer in 1962. He was a professor and then chairman of the Department of Civil Engineering at the University of Miami, Coral Gables, Florida, 1968-79. After joining UH, he served as the chairman of the Civil and Environmental Department, 1980-84, built a strong faculty and became the founding director of the Structural Research Laboratory, 1982-2003, which later bears his name. In 2005 he and his wife, Dr. Laura Ling Hsu, established the "*Thomas and Laura Hsu Professorship in Engineering*" at UH.

Dr. Hsu is distinguished by his research in construction materials and in structural engineering. The American Concrete Institute (ACI) awarded him its Wason Medal for Materials Research, 1965; Arthur R. Anderson Research Award, 1990 and Arthur J. Boase Award for Structural Concrete, 2007. Other national awards include the American Society of Engineering Education (ASEE)'s Research Award, 1969, and the American Society of Civil Engineers (ASCE)'s Huber Civil Engineering Research Prize, 1974. At UH, Professor Hsu's many honors include the Fluor-Daniel Faculty Excellence Award, 1998; Abraham E. Dukler Distinguished Engineering Faculty Award, 1998; Award for Excellence in Research and Scholarship, 1996; Senior Faculty Research Award, 1992; Halliburton Outstanding Teacher, 1990; Teaching Excellence Award, 1989.

In 2009, he was the honoree of the ACI-ASCE co-sponsored "*Thomas T. C. Hsu Symposium on Shear and Torsion in Concrete Structures*" on Nov. 9-10 at the ACI fall convention in New Orleans. The Symposium Volume, ACI SP-265, contains 29 papers presented by authors from around the world. In the same year, Houston Mayor Bill White proclaimed November 8, 2009, as *Dr. Thomas T. C. Hsu Day* in Houston for "bringing excellence and honor to the University of Houston and the City of Houston."

In 2010, Professor Hsu visited Taiwan and organized the "International Workshop on Infrastructure Systems for Nuclear Energy" at the National Center for Research on Earthquake Engineering (NCREE), on December 15-17. The 33 workshop papers given by world-leading experts will result in a 600-page book published by John Wiley and Sons, Ltd. He also gave two keynote speeches: One for the 10th National Conference on Structural Engineering, Chinese Society of Structural Engineering, on Dec. 1-3. The other was for the 4th Asian Concrete Federation Conference on Nov. 29.

Professor Hsu authored numerous research papers on shear and torsion of reinforced concrete and published three books: "*Unified Theory of Reinforced Concrete*" (Hsu and Mo, 2010), "*Unified Theory of Reinforced Concrete*" (Hsu, 1993) and "*Torsion of Reinforced Concrete*" (Hsu, 1984). Significant parts of Dr. Hsu's work on shear and torsion are codified into the ACI Building Code which guides the building industry in the USA.

Intrinsic to Dr. Hsu's work are two research innovations: (1) the concept that the behavior of whole structures can be derived from studying and integrating their elemental parts, or panels;

and (2) the design, construction and use of the *"Universal Panel Tester"* at UH, a unique, million-dollar test rig (NSF grants) that continues to lead the world in producing rigorous, research data on the constitutive models of reinforced concrete, relatable to real-life structures.

In his research on construction materials, Dr. Hsu was the first to visually identify micro-cracks in concrete materials and to correlate this micro-phenomenon to their overt physical properties. His research on fatigue of concrete and fiber-reinforced concrete materials made it possible to interpret the behavior of these structural materials by micro-mechanics.

Among his consulting projects, Dr. Hsu is noted for designing the innovative and cost-saving "*double-T aerial guideways*" for the Dade County Rapid Transit System in Florida; the curved cantilever beams for the Mount Sinai Medical Center Parking Structure in Miami Beach, Florida, and the large transfer girders in the American Hospital Association Buildings, Chicago, Illinois. He is currently a consultant to the US Nuclear Regulatory Commission (NRC).

Dr. Hsu is a fellow of the American Society of Civil Engineers and of the American Concrete Institute. He is a member of ACI Committee 215 (Fatigue), ACI-ASCE joint Committees 343 (Concrete Bridge Design) and 445 (Shear and Torsion). He had also served on ACI Committee 358 (Concrete Guideways), ACI Committee on Publication and ACI Committee on Nomination.

amplification factor for buildings designed by the equivalent static lateral force method. For buildings designed by elastic dynamic analysis, similar increases probably are not warranted.

Wall Shear Strength According to conventional U.S. practice, shear strength of a reinforced concrete wall is calculated for design purposes using equations that are effectively identical to those used for beams. Furthermore, it is common practice to assume both concrete and reinforcement contribute to strength, even in the plastic hinge zone. This practice is supported by experimental studies [Aktan and Bertero; 1985]. The practice of assuming the concrete contribution to shear strength is zero in the plastic hinge, as is done for beams and has been recommended by some researchers and designers for walls, will surely produce a more conservative design result, but for buildings where lateral drifts and plastic hinge rotations are reasonably controlled this approach does not seem warranted.

Because walls are primary lateral load resisting elements of a structural system, it should not be the designer's aim to make a wall as thin as practicable, but rather to construct it to be as sturdy as necessary. Experimental studies [Aktan and Bertero; 1985] indicate that seemingly brittle failure modes in the laboratory are possible due to web crushing when nominal shear stresses are high. With this aspect in mind, it is recommended that the maximum nominal shear stress in a wall under maximum expected shears should not exceed approximately $6\sqrt{f_c'}$ psi.

Summary Considering the results of the preceding paragraphs, it is recommended that where equivalent static analysis is used for wall design the design shear should be calculated according to Equation 8.

$$V_u = \omega \frac{M_o}{M_{code}} V_{code} \quad (8)$$

in which V_u = design shear, ω = amplification factor to account for the dynamic response phenomenon illustrated in Figure 25, M_o = expected wall plastic moment strength, M_{code} = wall base moment obtained from code lateral forces, and V_{code} = the shear obtained using the code lateral forces. The factor ω may be assumed equal to unity where dynamic analysis is used in design, and not less than 4/3 where static analysis is used in design.

CONCLUSION

Recent studies of the seismic response of reinforced concrete buildings have led to improved understanding of design requirements for wall buildings. Decisions made early in the design process regarding building proportions and, importantly, the ratio of wall area to floor plan area, significantly influence requirements for proportioning and detailing in later stages of design. Furthermore, the engineer can rely on simple tools in preliminary design stages to develop an understanding of how decisions on building proportion will influence subsequent detail requirements. After the preliminary design phase is completed, information regarding seismic demands on the building can be used to establish requirements for details and proportions of individual walls. In many cases, heavy transverse reinforcement required by current codes may be found to be unnecessary.

ACKNOWLEDGMENT

The work described in this paper was built out of collaboration with Professors P. Bonelli (Universidad de Santa Maria, Chile), R. Riddell (Pontificia Universidad Católica de Chile), M. Sozen (University of Illinois, USA), J. Wallace (Clarkson University, USA), J. Wight (University of Michigan, USA), and S. Wood (University of Illinois, USA) as part of a joint U.S.-Chile program on the 1985 Chile earthquake. The program was sponsored by the U.S. National Science Foundation.

REFERENCES

[ACI; 1989] *ACI Building Code Requirements for Reinforced Concrete* (ACI 318-89) (Revised 1992) American Concrete Institute, Detroit, Michigan

[Aktan and Bertero; 1985] Aktan, A. E. and Bertero, V. V., "RC Structural Walls: Seismic Design for Shear," *Journal of Structural Engineering*, ASCE, Vol. 111, No. 8, August 1985, pp. 1775-1791.

[ATC; 1978] "ATC-03-06, Tentative Provisions for the Development of Seismic Regulations for Buildings," Applied Technology Council, Palo Alto, California, 1978.

Ma, John

From: Ma, John
Sent: Saturday, June 11, 2011 9:07 PM
To: Tegeler, Bret; Thomas, Brian; [redacted]; [redacted]
Cc: Bergman, Thomas; Shuaibi, Mohammed
Subject: Two failure modes need to be investigated for AP1000 shield building due to aircraft impact
Attachments: Untitled

Bret:

During the June 6, 2011 DE meeting, Tom mentioned that the aircraft impact issue of the AP1000 shield building was still unresolved. I stated in my May 24, 2011 e-mail to you (attached) that I did not believe that the shield building wall with brittle modules can withstand aircraft impact because the brittle module used by Westinghouse has 25% less in strength, 300% less in ductility, and 400% less in energy absorption/dissipation capability, than its companion reinforced concrete module. Please make sure that Westinghouse does use or model this punching shear failure mode in its analysis because this is the governing (limiting) failure mode, not the ductile flexural mode as usually assumed by many.

Another failure mode of the shield building needs to be analyzed is that the aircraft hits the PCS tank on top of the shield building. The tank, the roof, and the compression ring girder are not only connected together but also provide stability for and among themselves. The tank supported by the roof and the compression ring girder is different from the tank supported on the ground. The difference is in the rotational stiffness at the connection. The AP1000 tank will rotate significantly during the aircraft impact because the rotational stiffness at the connection is relatively small compared to that while the tank is supported on the ground. I doubt that the connection has sufficient strength, stiffness, and ductility to resist that significant rotation of the tank during the aircraft impact. Once that connection is failed due to excessive rotation, the stability of the tank, the roof, and the compression ring girder is lost, and all will collapse onto the steel containment and crash it, and that may cause unintended accident.

Please make sure that Westinghouse will investigate these two failure modes.

John Ma

From: Ma, John
To: Tegeler, Bret
Cc: [redacted] Thomas, Brian; [redacted]
Date: Tuesday, May 24, 2011 10:28:00 AM
Attachments: image001.emz
image002.png
oledata.mso

Bret:

During this morning Branch meeting, Mohamed mentioned that you are at the Westinghouse to perform QA/QC related works, and aircraft impact is one of them.

As you may recall that I had told you and Brian several times that I was certain that Westinghouse's conclusion that the shield building could withstand the aircraft impact was incorrect had a correct analysis, including the adequacy of the computer code used, been conducted and evaluated. My assessment was based on that the computer code used by Westinghouse did not represent the nature of brittle failure of the out-of-plane shear tests, and instead of using ductile module behaviors. Aircraft impact on the shield building causes punching shear, which is also called radial shear around a punching type shear surface, or out-of-plane shear in a beam. As can be seen below, the brittle module used by Westinghouse has 25% less in strength, 300% less in ductility, and 400% less in energy absorption/dissipation, than its companion reinforced concrete module. Westinghouse claimed that the reason for them to switch from a reinforced concrete shield building in rev. 15 to SC modules in Rev. 16 is because that reinforced concrete could not take the aircraft impact, but the SC modules could. However, the test data indicate the opposite.

As you know that BP also raised questions on the aircraft impact issues on the AP1000. Brain told me before his rotation that he had asked you to coordinate with BP, me, and the contractor, who performed the aircraft impact review, to resolve the aircraft impact issues. I know that you have been busy and may not have had time to do that yet. Whenever you are ready, please let me know. We should correctly resolve the aircraft impact issue for the NRC and the public.

Take care of youself!

John Ma

a,c

Ma, John

From:	Chuang, Tze-Jer
Sent:	Tuesday, January 03, 2012 10:11 AM
To:	Ma, John
Subject:	FW: Site-Specific Extreme Wind Analysis for AP1000 Turkey Point COLA
Attachments:	ACI Code Concrete Punching Shear Strength.docx

From: Chuang, Tze-Jer
Sent: Tuesday, December 20, 2011 4:12 PM
To: [redacted] Tegeler, Bret; Valentin, Milton
Cc: Thomas, Brian; Shuaibi, Mohammed
Subject: RE: Site-Specific Extreme Wind Analysis for AP1000 Turkey Point COLA

ALL:

In this morning's discussion, we all agreed with shear stress of 89.15 psi due to impact of auto missiles in the case of DCD. We shifted our focus on the disagreement of ACI Code specified concrete dynamic shear strength. This led me to an in-depth investigation into ACI Code 349.

The attached document provides the appropriate Code sections that should be applied in analyzing this local punching shear problem. As can be seen, the concrete dynamic shear strength as given by ACI Code 349, is **112.77 psi**, regardless of one-way or two-way slabs or walls. The relevant ACI 349 sections will be provided to all of you for reference, if you are interested.

Jerry

From: [redacted]
Sent: Sunday, December 18, 2011 9:02 AM
To: Chuang, Tze-Jer; Tegeler, Bret; Valentin, Milton
Cc: Thomas, Brian; Shuaibi, Mohammed
Subject: RE: Site-Specific Extreme Wind Analysis for AP1000 Turkey Point COLA

Gentlemen – thanks for looking at this issue. Please let us meet in my office on Tuesday12/20 to discuss.

Mohamed

From: Chuang, Tze-Jer
Sent: Thursday, December 15, 2011 1:02 PM
To: Tegeler, Bret
Cc: [redacted] Thomas, Brian; Shuaibi, Mohammed; Valentin, Milton
Subject: RE: Site-Specific Extreme Wind Analysis for AP1000 Turkey Point COLA

Bret,

Thank you for your timely feedback. I am glad to know that your erroneous analysis is for internal use only. It is therefore very important to build a consensus on this issue within the SEB.

Since you indicated some disagreements in my CONCLUSION, I would like to know the specifics of your disagreements, and the bases to support your points. I suggest we discuss those in a weekly SEB Technical Discussion Meeting. .

Although I was not involved in the review, I'll be more than happy to help review RAI responses.

Jerry

From: Tegeler, Bret
Sent: Thursday, December 15, 2011 10:29 AM
To: Chuang, Tze-Jer
Cc: [redacted] Thomas, Brian; Shuaibi, Mohammed; Valentin, Milton
Subject: RE: Site-Specific Extreme Wind Analysis for AP1000 Turkey Point COLA

Jerry,
Thanks for your commentary. While I disagree with some of your conclusions, the bottom line is that the applicant will make the safety case. Milton and I premised the calculation by stating that the intent was not to supplant the need for the applicant to justify the design. I am sure you can bring your insights to bear when you review the response.

Nice job.
Bret

From: Chuang, Tze-Jer
Sent: Thursday, December 15, 2011 9:51 AM
To: Tegeler, Bret; Valentin, Milton
Cc: [redacted] Thomas, Brian; Shuaibi, Mohammed
Subject: RE: Site-Specific Extreme Wind Analysis for AP1000 Turkey Point COLA

Bret and Milton:

<u>SYNOPSIS of TECHNICAL ISSUE and POTENTIAL SAFETY CONCERN</u>

Auto missiles generated by extreme hurricane wind at the Turkey Point site have potential safety concern for Seismic Category I structures in creating huge openings/holes by local punching shear, thereby compromising structural integrity. The above-referenced analysis using [ ]a,c is non-conservative and needs be justified. Re-analysis using method consistent with test data gave rise to higher stresses resulting in noncompliance with respect to ACI code allowable, thus, not meeting GDC 2 and GDC 4 in Appendix A to 10 CFR Part 50. Mitigation strategy such as re-designing or re-analyzing the building walls may need to be implemented to resolve this safety issue.

<u>DISCUSSION</u>

Thank you for the courtesy of sending me the above-referenced analysis. The objective of the analysis was to find out that, under the condition of the site-specific wind speed at Turkey Point exceeding the standard design- basis specified in the Ap1000 DCD, whether or not the standard design of AP1000 nuclear island structures can withstand the auto missile attack and maintain their structural integrity. This issue was first raised by John Ma during his review on shield building, and I had briefed SEB staffers and management on this issue which resulted from my DCD 3.5.1.4 and 3.5.3 review. This safety issue in the DCD review space was resolved based on TR133, Rev. 1 which was audited by me and found acceptable. This report, titled:"<u>Nuclear Island Tornado Missile Auto Impact above 30 feet of Grade</u>" (APP-1000-CCC-015) should be used as a base-line in your analysis and be included in the reference section on Page 3.

There are two cases analyzed in your report: (1) DCD missile speed of 105 mph; (2) Turkey point missile speed of 180 mph. I first checked your Case (1) shear stress results on Page 8: 23 psi < allowable [ ]a,c against Case (1) results for base-line DCD: 89.15 psi < allowable 112.77 psi. I found your calculated concrete allowable shear stress of [ ]a,c is very close to DCD's [ ]a,c and is acceptable, although John may disagree on this shear capacity (For the purposes of this discussion, let's assume this is correct). However, your calculated shear stress of 23 psi is very low in comparison with

DCD's 89.15 psi based on same missile speed of 105 mph. This prompted me to dig into your analysis in order to find the root causes of this shear stress discrepancy.

After an in-depth investigation into your methodology, I found you assumed rectangular impulse load function for impact loading history to obtain the peak force and impact duration **for the given total impulse,"I"** (see Page 4 and Figure 1 on Page 7).On the other hand, the DCD based on TR 133 used **actual test data** resulting from dynamic tests performed by Southwest Research Institute. The loading history curve from the tests give rise to realistic value of peak load directly, rather than assumed. Based on this comparison, I believe your basic assumption of loading is non-conservative, and the resulting shear stresses locally should be around 89.15 psi, not in the vicinity of your calculated value of 23 psi. I found the results in TR 133 were acceptable because they are based on technically sound methodology, whereas your assumed impact loading deviated substantially from the test data.

CONCLUSION

The Case (1) DCD has a safety margin of about 20% on the issue of local punching shear (i.e., 89 psi versus allowable 112 psi). In the case of Turkey Point, Case (2), which almost doubled the loading level [ ] a,c **I am afraid** the true shear stress (estimated at about [ ] a,c could significantly exceed the allowable (112 psi), and it could potentially become a safety issue. Accordingly, a re-analysis using methods consistent with the test data to assure structural integrity is warranted. Hope that your preliminary analysis has not yet been released to others outside the SEB. Please feel free to get back to me, should you have questions or need help.

Jerry

From: Valentin, Milton
Sent: Tuesday, December 13, 2011 2:00 PM
To: [redacted] Chuang, Tze-Jer; Park, Sunwoo; Kazi, Abdul
Subject: FW: Hurricane Missile Calc

FYI - SRP 3.5.3 evaluation is attached.

From: Tegeler, Bret
Sent: Friday, December 09, 2011 8:14 AM
To: Chakrabarti, Samir
Cc: [redacted] Valentin, Milton
Subject: Hurricane Missile Calc

Samir,
Attached is a draft what Milton and I developed for Turkey Point/Levy. Perhaps it may be of use in your assessment of STP? I am around today if you need anything....
Bret

Bret Tegeler, Sr. Structural Engineer
U. S. Nuclear Regulatory Commission
Mail Stop T-10H9
Washington, DC 20555-0001
(301) 415-6793

Rebuttal to the Staff Response to Dissenting View on the SER on the Design of the AP1000 Shield Building

by

John S. Ma, Ph.D., P.E.
Recipient of ACI Raymond C. Reese Structural Research Award Medal
NRC/NRO/DE/SEB1

October 22, 2010

Introduction

The author has been attending a mandatory training course. Due to the time constraint, the author chooses the two most fundamental flaws, which appeared to be the basis for the staff SER, in the staff's response for rebuttal in an effort to cause the staff's re-examination of the adequacy of its SER.

Executive Summary

The author reviewed the staff response to the author's dissenting view, dated October 17, 2010, and found that the staff believed that (1) the ACI Code design allows brittle structural components, and (2) WEC demonstrated that the shield building as designed has ductility where it is needed and sufficient strength in areas where the structure is brittle and in those brittle areas the structure will not be subject to loads beyond that of the design basis loads. The staff's finding appeared to have based on the staff's reliance on WEC computer codes analysis results without carefully examining the applicability or inapplicability of the code to the complex irregular configurations of the shield building. These beliefs appeared to have formed the basis of the staff SER. The author will demonstrate that the staff beliefs are false.

The reevaluation of tornado missiles and aircraft missiles for the SC wall taking into account of the brittle nature of module #2 has not been performed and is needed if module #2 is allowed to be used, and the associated difficulties with that reevaluation and justification are mentioned.

<u>Staff belief Number 1</u>

The staff response states,

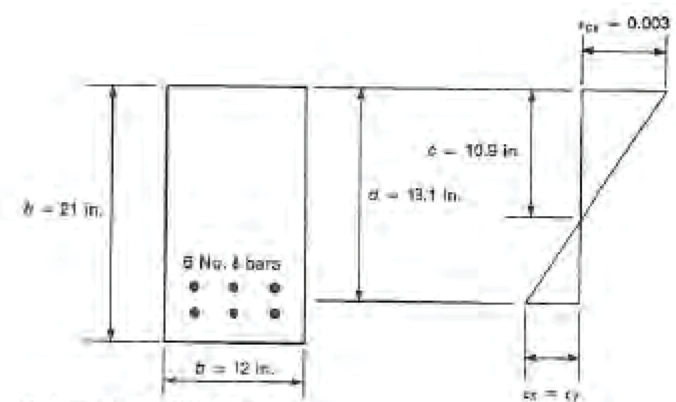
> The staff believes that the design of the shield building embraces the underlining engineering principles of the ACI Code by using the code formulas for strength and ductility in the design. The Code does not specify a ductility level nor does it specify that ductility should be in every single structural component of the structure.

<u>The author's conclusion</u>

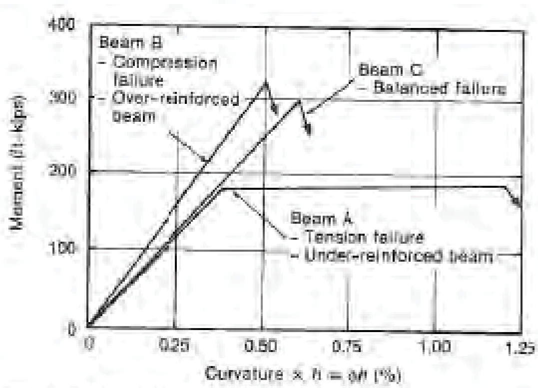
The author will demonstrate that the ACI Code requires not only every structural component to be ductile but also all sections within the structural component to be ductile as well. The author will present the Code required minimum ductility level for sections of a beam in a building located in non-seismic zones, which does not need an earthquake-resistant design in accordance with the Chapter 21 of ACI 318-08 Code, to demonstrate that ductility is embedded in the Code although it is not specified in plain English language.

1. The staff states in page 5 “The Code does not specify a ductility level nor does it specify that ductility should be in every single structural component of the structure.”

Every single structural component, which is referred to in the staff’s statement, can be a beam, a column, a floor, and a wall. A beam consists of (cross) sections within the length of the beam, and the design of a section, based on the ACI Code, will be used as a demonstration that the minimum required ductility level is embedded in the Code and can be calculated. The author uses the diagram below, which is excerpted from a text book, to help explain that ductility is not only the fundamental requirement but also the starting point of a design for any reinforced concrete structures.



(c) Beam C—Balanced failure



(d) Moment-curvature diagram

In the case of beam C, the strain distribution at failure, shown in Fig. 4–16c, involves simultaneous crushing of the concrete and yielding of the steel. This case, which also exhibits a brittle failure as shown in Fig. 4–16d, marks the boundary between ductile tension and brittle compression failures—hence the name *balanced failure*.

The above diagram illustrates the basic concept of the ACI Code design for a section of a beam, which is subjected to bending (flexure). The diagram consists of three sections representing three beams. The only variable or difference among the three sections or beams is the amount of steel reinforcement: 3-#8 for beam A with area of steel equal to 2.37 square inch (3 times 0.79), 8-#8 for beam B with area of steel equal to 6.32 square inch (8 times 0.79), and 6-#8 for beam C with area of steel equal to 4.74 square inch (6 times 0.79), and other parameters remain identical for the three sections and beams. The section in a beam is classified into three different conditions depending on the design: over-reinforced section, balanced section, and under-reinforced section. The beam will fail in two different modes: brittle for the over-reinforced and the balanced sections, and ductile for the under-reinforced section, as explained in the paragraph just below the diagram. The physical conditions of the over-reinforced section, the balanced section, and the under-reinforced section, are explained below:

- The balance failure represents the condition that the maximum strain at the extreme compression fiber just reaches 0.003 simultaneously with the first yield strain in the tensile reinforcement.
- The over-reinforced failure represents the condition that the maximum strain at the extreme compression fiber reaches 0.003 while the tensile reinforcement has not reached yield strain.
- The under-reinforced failure represents the condition that the maximum strain at the extreme compression fiber has not reached 0.003 while the steel reinforcement has reached yield strain.

As can be seen in the above diagram, the over-reinforced section has the highest strength, the under-reinforced section has the lowest strength, and the balanced section has the strength in between; however, both the over-reinforced section and the balanced section failed in brittle modes and only the under-reinforced section behaves and eventually fails in a ductile manner. The ductility ratio of the balanced section with respect to the yielding of steel is 1.0, because the strain of the steel at failure of the section equals to the yield strain of the steel. The ductility ratio of the over-reinforced section is less than 1.0. Although both the over-reinforced and balance sections posses higher strength than that of the under-reinforced section, the ductility and toughness or energy dissipation capability, the area under the curve, (see definitions in Figure 1 of the author's write-up, dated September 27, 2010) of the under-reinforced section is much greater than that of the over-reinforced and balanced sections (the area under the straight line). These phenomena are part of reasons that the ACI Code prohibits the design of over-reinforced and the balanced sections and only allows the design for under-reinforced sections for beams.

ACI Code classifies structures into six (6) categories (See Section 1.1.9.2 of ACI 318-08 code) from A thru F. Structures, which are located in non-seismic zones, and do not need a seismic design is classified as category A (See Section 1.1.9.2 of ACI 318-08 code). All structures in other categories (B thru F) need to be designed in accordance with Chapter 21, "Earthquake-Resistant structures" of the Code. Even for category A structures, the Code requires all sections of a beam to achieve a minimum ductility level. This minimum ductility level is achieved by specifying that the designed section should not fail in concrete crushing until the steel reinforcement has been yielded to a strain of 0.005, (See Section10.3.4 of ACI 318-08 Code). For the steel reinforcement with yield strength of 60 ksi, the yield strain is 0.002 (60 ksi divided by the modulus of elasticity of 30,000). Therefore, the ductility ratio of the section with respect to steel reinforcement yielding is **2.5** (0.005 divided by 0.002). For sections that have high stresses and may form a plastic hinge with possible stress redistribution, the Code requires that the concrete crushing in those sections (regions) should not occur until the steel reinforcement strain reaches 0.0075 (see Section R10.3 of the ACI 318-08 Code). The ductility ratio for these sections with respect to steel reinforcement yielding is **3.75**. For a category A building, as long as the beams meet the minimum ductility ratio as stated above, there is no need to consider a design for energy dissipation capability (toughness), because the design is mainly to resist **static** forces (vertical loads or weights), and, as long as the structural members have sufficient strength to resist the applied static loads, the building will be safe. Therefore, the use of demand to strength criteria, which is used by the staff as the criteria for the evaluation of the shield building design, in conjunction with the code specified built-in minimum ductility in sections of members is sufficient.

For earthquake-resistant buildings, which are classified from B thru F, ACI 318-08 Code, Chapter 21, Earthquake-Resistant Structures, Section R21.1.1, states,

> The design and detailing requirements should be compatible with the level of energy dissipation (or toughness) assumed in the computation of the design earthquake forces. The terms “ordinary,” ”intermediate,” and “special” are specifically used to facilitate this compatibility. The degree of required toughness and, therefore, the level of required detailing, increases for structures progressing from ordinary through intermediate to special categories. It is essential that structures assigned to higher SDCs (seismic design category) posses a higher degree of toughness.

Therefore, for earthquake-resistant buildings, the emphasis of design is in **energy dissipation capability**. This is because ground motions from earthquakes impart energy into buildings, and the energy needs to be dissipated from the building, and the means to dissipate that energy is through **ductility** of the structure. In Section 3.2 of the author’s write-up, dated September 27, 2010, the conversion of energy dissipation capability to ductility was described. The staff did not embrace this portion of the code requirement for its review on the SC wall.

From the above discussion, it is clear that ACI Code even specify minimum ductility requirements for sectional design of reinforced concrete structural components in ordinary Category A structures that are located in non-seismic zones, which do not require seismic design. For earthquake-resistant structures, Categories B thru F, the Code increases the required ductility progressively as the category of structures advances from B thru F, but the staff did not address it in its SER.

2. The staff response states “The staff believes that the design of the shield building embraces the underlining engineering principles of the ACI Code by using the code formulas for strength and ductility in the design.”

The above statement indicates that the staff believes that the shield building design has used ACI Code formulas for ductility. How could the staff believe that “the Code does not specify a ductility level nor does it specify that ductility should be in every single structural component of the structure” on one hand, and that the shield building design has used ACI Code formulas for ductility on the other hand while the two beliefs should be mutually exclusive? Contrary to the staff statement that the design of the shield building embraces the underlining engineering principles of the ACI Code by using the code formulas for ductility in the design, the shield building has never been designed for the ACI Code ductility formulas. The author would like to see that evidence if it exists. Since there was no formulas or proven analytical methods for ductility design applicable to SC wall modules, WEC proposed to use ductility testing for SC modules to demonstrate their acceptability and the NRC agreed to that approach because there was and still is the only way to demonstrate ductility for SC modules.

Staff belief Number 2

The staff response states,

> The staff finds that WEC demonstrated that the shield building as designed has ductility where it is needed and sufficient strength in areas where the structure is brittle and in those brittle areas the structure will not be subject loads beyond that of the design basis loads.
>
> To assess the behavior of the shield building for collapse, WEC performed a nonlinear push-over analysis. The push-over method is an accepted industry practice for estimating the limit state (i.e., collapse load) and corresponding modes(s) of failure of a structure due to seismic loading. The staff finds that the push-over analysis model used by WEC was validated by comparison with relevant experiments (in-plane, and out-of-plane shear) and is therefore acceptable for estimating the response of the shield building to severe seismic demands.

The author's conclusion

The author finds that the staff's finding was based on its reliance on the results of the push-over computer code analysis. The author had concluded in Section 9.4 in his previous write-up, dated September 27, 2010, and NII of UK has recently concluded, that the push-over analysis is not applicability to the complex irregular configuration of the shield building. Therefore, the staff's finding is questionable at the best and may lead to the wrong conclusion with false confidence. While the staff has shown a great confidence in, and reliance on, WEC's computer analysis and their results, NII, BNL, and the author, have discovered many problems with WEC's analysis and results. The supporting materials to the author's and NII's conclusion and many other issues related to the analysis and design of the shield building are stated or listed below.

The staff first statement above is a new revelation without stating its basis for the finding. On what basis does the staff know which portion of the shield building needs ductility, and other portions do not, and that in those brittle areas the structure will not be subject to loads beyond that of the design basis loads? The author has demonstrated above that the ACI Code only allows the design of ductile sections, and therefore the staff's acceptance for brittle areas in the shield building is contradictory to ACI Code requirements.

With respect to the staff finding on the ability of properly located ductile and brittle areas in the shield building SC wall and of the magnitude of loads in the brittle areas, it appears that this finding was relied upon the push-over computer analysis as stated in the second statement above. With respect to the applicability of the push-over analysis for the shield building, NII of UK asked in question no. 75, "Explain the use of pushover analysis, which is only appropriate for simple regular structures that respond in the first mode (*The author notes that the first mode is a cantilevered beam with a concentrated force applied to the top of the beam and the shape of that deflection curve is call the first mode, which is the shape assumed by WEC in its September 3, 2010 submitted),* and concluded in question 75, "The use of pushover analysis to judge the performance of a complex structure responding in many modes is inappropriate." (All NII questions can be seen in the attached file). The author also explained in Section 2.2, Major problems of the SC wall, under subtitle, "Analysis" on the problems of analysis resulting from the complex irregular configurations of the shield building and concluded the same inapplicability of the WEC push-over analysis for the shield building for the same reason as stated in Section 9.4 of the author's previous write-up, dated September 27, 2010.

The staff finding stated that the magnitude of loads and strengths in different areas of the shield building wall were properly demonstrated by WEC. However, evidence was not provided to substantiate that finding. NII has problems with the WEC stated strength, ductility, stress, analysis methods, and design methods, especially in shears in the SC wall and for tie-bars, as stated in the following questions. With respect to the problems related to strength, stress, and ductility, NII Question No.84 states, "The statement that the Shield Building is [ ][a,c] makes no sense.," and "WEC note that [ ][a,c] and [ ][a,c] On what basis are the statements regarding ductility made? What are lower stresses?" Question 107 states "How is it possible to sustain such a high stress in concrete if nonlinear constitutive behaviour has been considered? There are also large areas shown with tensile stress in the order of [ ][a,c] Similar comments apply to Figures B-98 to B-103, B-140 to B-145 and B-188 to B-192". With respect to the design procedures, NII question Nos. 55 and 60 states, "What is the design procedure for out-of-plane shear where the loading effects include SSE shaking? How is interaction between in- and out-of-plane shear considered?" and "Consideration to combined in- and out-of-plane shear should be given in the development of the acceptance criteria. The average shear stresses at the onset of cracking should be reported." With respect to failure modes of the SC wall, NII question No. 62 states, "The results of the tests indicate that a brittle failure plane across the top of the shear studs is potentially the governing failure mode rather than a ductile failure of the studs themselves. However, there is no discussion in this document of potential failure modes associated with longitudinal failure planes in the concrete itself. Checks undertaken to BS5400 Part 5 Cl. 6.3.3.2 indicate that this mechanism controls the capacity due to an absence of rebar crossing the critical longitudinal shear plane across the top of the studs." NII question 100 states, "Checks should be performed for diagonal tension, diagonal compression and shear friction. What is the limit for shear friction? What is the limit for diagonal compression?" With respect to the WEC tie-bar design, NII question No. 8 states,

> "In the design of the enhanced shield building the 0.75inch diameter tie bars are spaced at a maximum of [ ][a,c] centres and span transversely between the faces of the steel plates. We have concerns that various structural roles they perform are not properly reflected in the design methodology. We have identified four co-existent loading mechanisms that we believe will generate stresses in the bars that in our opinion need to be addressed in any rational design methodology.
>
> <u>Wet Concrete Pressure</u> - during casting the tie bars will experience tensile stresses due to the wet concrete pressures developed behind the steel plates. These tensile stresses will be locked into the tie bars post-curing and should be accounted for in the final design.
>
> <u>Out-of-Plane shear</u> - the tie bars will be mobilised in tension when resisting out-of-plane shear in a similar manner to shear stirrups in conventional reinforced concrete design
>
> <u>Interface shear</u> – longitudinal shear stresses generated at the interface between the steel plates and the concrete core will be resisted by the combined action of the shears studs and tie bars.

Splitting forces – there is evidence that splitting forces are generated at the connections with RC and elsewhere that create bursting stresses in the concrete leading to splitting failure without the presence of transverse ties.

We require a detailed explanation justifying why it is appropriate for these load actions to be treated independently when determining the utilisation of the tie bars.”

With respect to problems related to ductility, NII question No. 75 states,” Explain why ductile limit states govern after initial yielding. What is the failure hierarchy in the ESB? Identify which ductile limit states enable the ESB to achieve deformations that are 3 to 4 times the SSE deformations. Demonstrate how system behaviour at RLE shaking is governed by ductile limit states.” And NII question states, “Why is a ductility factor imposed if the failure mode can change from ductile to brittle through the use of excessive reinforcement? The second paragraph in this section is not clear. Why will out-of-plane shear not control the margin and what “shear stress exceeds yield”? A “margin factor” calculation should consider both normal forces acting on the plate, not just the circumferential force. NII question No. 102 states, ” Why is a ductility factor imposed if the failure mode can change from ductile to brittle through the use of excessive reinforcement? The second paragraph in this section is not clear. Why will out-of-plane shear not control the margin and what “shear stress exceeds yield”? A “margin factor” calculation should consider both normal forces acting on the plate, not just the circumferential force.”

With respect to problems related to computer codes, NII question No. 128 states,

“It is noted in Figure 8.7-7 and 8.7-15 that the test results exhibit significant and rapid unloading, but the FE analysis does not.

This discrepancy is poorly excused by the preamble in section 8.5, p 8-17 (see point 121).These tests seem to indicate a poor agreement between testing and analysis, but this is not discussed. In 8.7-7 a kinetic energy jump has been identified close to the test failure point, notwithstanding that the structure continues to have a positive gradient on the load-deflection plot.

When acting as part of a larger model, the important behavior will be the actual load deflection plot and not the existence of a small kinetic energy jump. We are concerned that the existence of this kinetic energy jump might be used to justify some sort of correlation between test and analysis. But if the curve does not follow the test then the correlation does not exist. Please see point 121. Several other problems related to WEC computer code analysis problems can be seen in the attached file.”

BNL performed a detail review on the WEC analysis by LS-DANA computer code for the shield building, and concluded that “Based on the detailed evaluation conducted, there is no confidence that an appropriate level of quality assurance was implemented in the conduct of the LS-DYNA analyses”. The report is attached.

The need and problem of reevaluation of the SC wall for tornado missiles and aircraft missiles

The reviewer on tornado missile for the AP1000 stated that his review for SRP Section 3.5, tornado missiles, is only applicable to concrete structures (See attached e-mail) and not to the SC wall. The lead reviewer for AIA indicated that a review for aircraft missiles taking into account of the brittle material of module #2 is required (see attached e-mail). The author doubts that such a review can be easily done because there is no formula for missiles with this type of brittle material characteristics. The next problem is how the staff justifies that the ability of the SC wall resisting missiles can be much lower than a companion reinforced concrete shield building wall.

NEHRP (National Earthquake Hazards Reduction Program) Recommended Seismic Provisions

for New Buildings and Other Structures (FEMA P-750)

2009 Edition

Prepared for the Federal Emergency Management Agency of the
U.S. Department of Homeland Security
By the Building Seismic Safety Council of the
National Institute of Building Sciences

BUILDING SEISMIC SAFETY COUNCIL
A council of the National Institute of Building Sciences
Washington, D.C.
2009

Resource Paper 2

NONLINEAR STATIC PROCEDURE

This resource paper was prepared by Technical Subcommittee 2, Design Criteria and Analysis and Advanced Technologies as a replacement for the Appendix to Chapter 5 of the 2003 edition of the NEHRP Recommended Provisions. It revises the information on the nonlinear static procedure (NSP) to allow its use in design of regular buildings less than 40 feet in height. The principal value of this approach as currently presented is for the design of buildings that are controlled by drift limits. Such buildings can be designed to have sufficient stiffness without using the equivalent lateral force (ELF) procedure and to have sufficient strength without conducting detailed member evaluations ($R_d < R/\Omega_0$). In the future, the height limitation may be relaxed if, for example, the NSP is used in conjunction with a nonlinear dynamic analysis.

Because requirements for the nonlinear static procedure are now specified in ASCE/SEI 41-06, it is simpler to refer to that document than to write applicable requirements into the Provisions. Modifications to the ASCE/SEI 7-05 requirements are introduced here to maintain consistency with the nonlinear static procedure information presented in the 2003 Provisions.

The 40-foot height limit was selected based on the accuracy of response quantities determined for a three-story moment-frame structure; no height limit was identified in the FEMA-funded Applied Technology Council project on the evaluation of inelastic seismic analysis procedures (Improvement of Nonlinear Static Seismic Analysis Procedures, FEMA 440). Although higher modes will have a similar influence on ELF quantities, the higher base shear strengths and story shears of the ELF procedure will tend to result in smaller member ductility demands. Thus, precision in the NSP estimates is especially important when system strengths are lower than those resulting from use of the ELF approach, which evaluates member deformation demands in detail.

This resource paper simplifies the language used to establish whether lateral strength is nominally less than that required by the ELF procedure. This is now stated succinctly as $R_d > R/\Omega_0$. Section references have been harmonized with ASCE/SEI 7-05 section numbers. If adopted for ASCE/SEI 7-10 or subsequent editions, the chapter number assigned to the requirements portion of this paper will have to be substituted where "X" appears below.

REQUIREMENTS

X Nonlinear Static Procedure

X.1 Definitions

Target Displacement. An estimate of the maximum expected displacement of the control node, determined according to Section 3.3.3.3.2 of ASCE/SEI 41 Supplement1 using S_a defined as a design earthquake spectral response acceleration according to the 2009 *NEHRP Recommended Seismic Provisions* at the effective period.

X.2 Notation

Q_{Ei} Force in i^{th} member determined according to Section 12.15.8.
R_d The system strength ratio as determined by Equation X-1
R_{max} The maximum strength ratio, defined by Equation 3-16 of ASCE/SEI 41 Supplement 1
γ_i The deformations for member i.
Ω_0 See Section 11.3.

X.3 Applicability. Regular structures less than 40 feet in height in Occupancy Categories I and II may be designed using the nonlinear static procedure following the requirements of this chapter.

X.4 Seismic-force-resisting System. The seismic-force-resisting system shall conform to one of the types in Tables 12.2-1 and 15.4-1 and shall be in accordance with the seismic design category and height limitations indicated in these tables. The appropriate response modification coefficient, R, and system overstrength factor, Ω_0, identified in these tables shall be used, subject to the additional requirements of this chapter.

X.5 Modeling and Analysis. Modeling and analysis shall conform to Section 3.3.3 of ASCE/SEI 41 Supplement 1 except that: (a) S_a shall be defined as a design earthquake spectral response acceleration according to the *NEHRP Recommended Seismic Provisions* at the effective period and (b) the analysis shall be conducted for seismic actions occurring simultaneously with the effects of dead load in combination with not less than 25 percent of the required design live loads.

COMMENTARY

This resource paper presents proposed requirements for nonlinear static analysis, a seismic analysis procedure also sometimes known as pushover analysis, for review and comment and for adoption into a subsequent edition of the *NEHRP Recommended Provisions*.

Although nonlinear static analysis has only recently been included in design provisions for new building construction, the procedure itself is not new and has been used for many years in both research and design applications. For example, nonlinear static analysis has been used for many years as a standard methodology in the design of the offshore platform structures for hydrodynamic effects and has been adopted recently in several standard methodologies for the seismic evaluation and rehabilitation of building structures, including the *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings* (FEMA 350, 2000), *Seismic Rehabilitation of Existing Buildings* (ASCE/SEI 41-06, 2007), and *Seismic Evaluation and Retrofit of Concrete Buildings* (Applied Technology Council, 1996). Nonlinear static analysis also forms the basis for earthquake loss estimation procedures contained in the earthquake module of the multihazard software application HAZUS-MH MR2 (FEMA, 2006) and its Advanced Engineering Building Module (FEMA, 2002). A critical review of and improvement to nonlinear static analysis methods, *Improvement of Nonlinear Static Seismic Analysis Procedures*, was published as FEMA 440 in 2005. Although it does not explicitly appear in the *Provisions*, the nonlinear static analysis methodology also forms the basis for the equivalent lateral force procedures contained in the provisions for base-isolated structures and structures with dampers.

One of the controversies surrounding the introduction of this methodology into the *Provisions* relates to the determination of the limit deformation (sometimes called a target displacement). Several methodologies for estimating the amount of deformation induced in a structure as a result of earthquake ground shaking have been proposed and are included in various adoptions of the procedure. The approach presented in this paper is based on statistical correlations of the displacements predicted by linear and nonlinear dynamic analyses of structures recommended in the FEMA 440 report (2005) on the evaluation of inelastic seismic analysis procedures.

A second controversy relates to the limited availability of consensus-based acceptance criteria to be used to determine the adequacy of a design once the forces and deformations produced by design earthquake ground shaking are estimated. It should be noted that this limitation applies equally to the nonlinear response history approach, which already has been adopted into building codes.

A third controversy relates to the effects of higher modes (or multi-degree-of-freedom effects for structures responding nonlinearly) on response quantities. FEMA 440 identifies significant disparities between response quantities determined by nonlinear static analysis and those determined by nonlinear dynamic analysis for all but low-rise structures; therefore, use of the nonlinear static procedure for the design of members proposed here is limited to structures 40 feet or less in height. This limitation has resulted in the nonlinear static procedure being located in Part 3 of the *Provisions*. The nonlinear static procedure may be used to ensure that structures designed according to the equivalent lateral force procedure achieve strengths comparable to code expectations. Interstory drifts are compared with tabulated allowable story drifts to maintain consistency with past practice, although it is recognized that larger interstory drifts should be anticipated due to higher mode or multi-degree-of-freedom effects.

Nonlinear static analysis provides a simplified method of directly evaluating nonlinear response of structures to strong earthquake ground shaking that can be an attractive alternative to the more complex procedures of nonlinear response history analysis. It may be useful for characterizing system strength and stiffness and for establishing that the structure develops a desirable inelastic mechanism.

REFERENCES

American Society of Civil Engineers/Structural Engineering Institute. 2006. *Seismic Rehabilitation of Existing Buildings*, ASCE/SEI 41-06, with Supplement 1. American Society of Civil Engineers. Reston, Virginia.

Applied Technology Council. 1996. *Seismic Evaluation and Retrofit of Concrete Buildings*, SSC Report 96-01. Seismic Safety Commission, State of California, Sacramento, California.

Applied Technology Council. 2005. *Improvement of Nonlinear Static Seismic Analysis Procedures*, FEMA 440. FEMA, Washington, D.C.

Building Seismic Safety Council. 2003. *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings and Other Structures*, FEMA 450. FEMA, Washington, D.C.

The standard addresses these objectives by requiring that each structure be assigned to one of the four occupancy categories presented in Chapter 1 and by assigning an importance factor to the structure based upon that occupancy category. (The two lowest categories, Ordinary and Low Hazard, are combined for all purposes within the seismic provisions). The occupancy category is then used as one of two components in determining the Seismic Design Category (see Section C11.6) and is a primary factor in setting drift limits for building structures under the design earthquake ground motion (see Section C12.12).

Figure C11.5-1 shows the combined intent of these requirements for design. The vertical scale is the likelihood of the ground motion with the MCE being the rarest considered. The horizontal scale is the level of performance of the structure and attached nonstructural components from collapse prevention at the low end to operational at the high end. (These performance levels are discussed further at other locations in the commentary.) The basic objective of collapse prevention at the MCE for ordinary structures (Occupancy Category II) is shown at the lower right by the solid triangle; protection from life-threatening damage at the design ground motion (defined by the standard as two-thirds of the MCE) is shown by the open triangle. The performance implied for higher occupancy categories is shown by square and circles. The performance anticipated for less severe ground motion is shown by dotted symbols. The three (net) classes and the numerical values assigned are far too coarse to assure the portrayed outcome for all structures, but it is judged to be adequate for the purpose given present limitations of knowledge and tools.

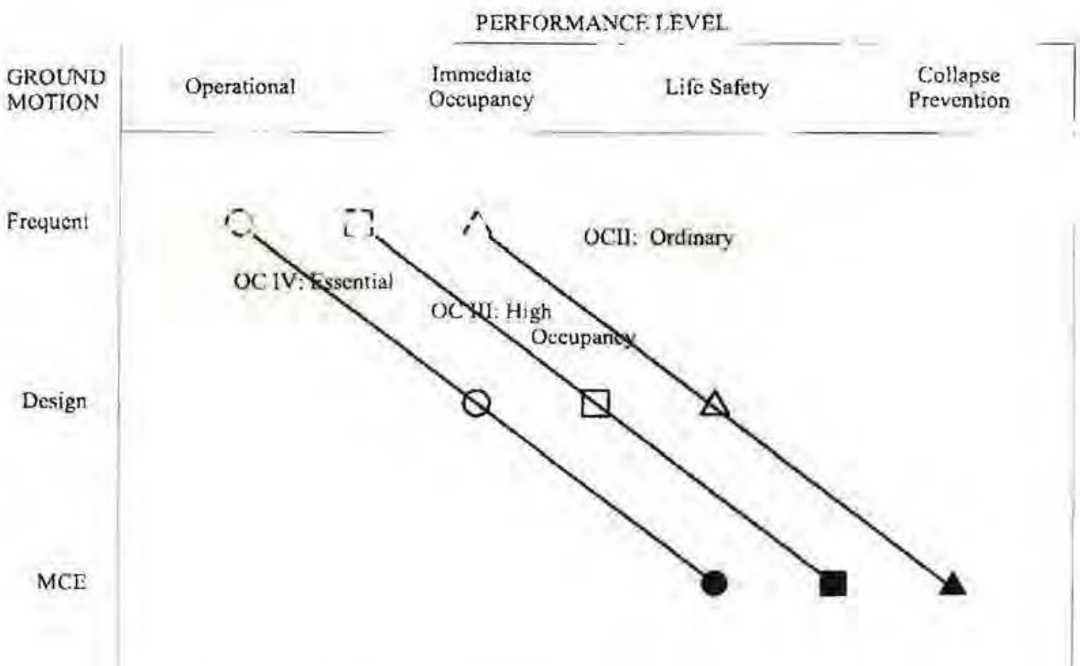


Figure C11.5-1 Expected performance as related to occupancy category (OC) and level of ground motion.

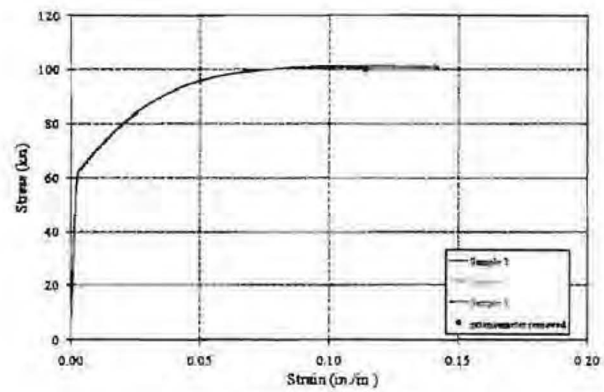
C11.5.1 Importance Factor. The importance factor is used throughout the standard in quantitative criteria for strength. In most of those quantitative criteria, the importance factor is shown as a divisor on the factor R or R_p in order to send a message to designers that the objective is to reduce damage for important structures in addition to preventing collapse in larger ground motions. The R and R_p factors adjust the computed linear elastic response to a value appropriate for design; in many structures, the largest component of that adjustment is ductility (the ability of the structure to undergo repeated cycles of inelastic strain in opposing directions). Inelastic strain damages a structure so, for a given strength demand, reducing the effective R factor (by means of the importance factor) increases the required yield strength, thus reducing ductility demand and related damage.

C11.5.2 Protected Access for Category IV Structures. Those structures considered essential facilities for response and recovery efforts must be accessible to carry out their purpose. For example, if the collapse of a simple canopy at a hospital could block ambulances from the emergency room admittance area, the canopy must meet the same structural standard as the hospital. This requirement must be considered in the siting of essential facilities in densely built urban areas.

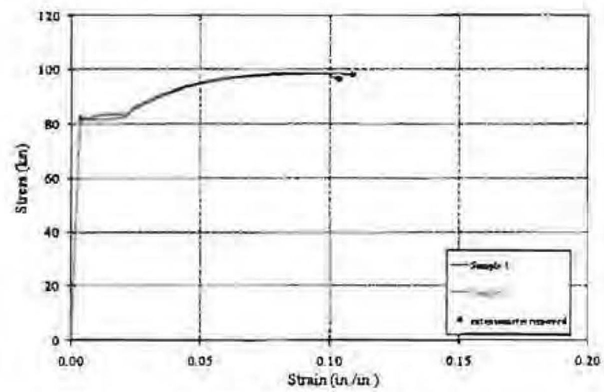
A.3 ASTM A615 Material Properties

Table A4 Summary of A615 tension properties.

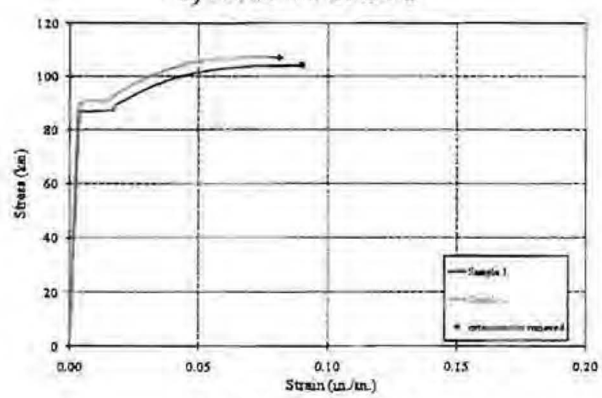
bar size		#4	#4	#4	#4	#4	#3	#3	#3	#3
batch		1	2	3	4	5	1	2	3	4
lab		UC	UC	UC	UC	PITT	UC	UC	PITT	PITT
# of samples		3	2	2	4	3	3	2	3	3
f_u	ksi	100.7	98.4	105.4	105.0	102.3	100.4	102.7	103.0	107.8
ε_u		n.r.	n.r.	n.r.	n.r.	0.205	n.r.	n.r.	0.153	0.133
E_{calc}	ksi	26934	25851	27596	23945	28635	29492	27268	29124	26178
f_y based on…										
$\varepsilon = 0.0035$	ksi	62.6	81.4	86.3	83.4	61.6	55.4	63.5	67.3	69.1
$\varepsilon = 0.0050$	ksi	64.2	81.9	88.2	92.9	64.3	65.3	63.5	68.4	73.0
$\varepsilon = 0.0070$	ksi	66.6	81.6	88.8	93.5	66.3	65.5	64.3	69.3	73.7
$\varepsilon = 0.0100$	ksi	70.1	82.4	88.7	90.3	69.3	67.8	67.9	70.6	75.1
0.2% offset	ksi	63.5	81.5	88.2	90.2	61.5	65.1	63.6	67.3	69.1



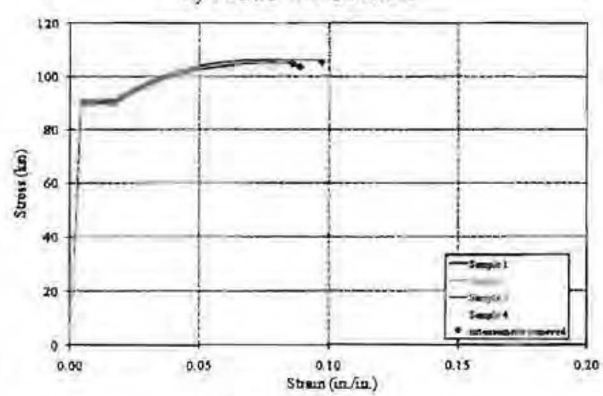
a) A615 #4 batch 1



b) A615 #4 batch 2



c) A615 #4 batch 3



d) A615 #4 batch 4

Figure A3 Axial stress-strain curves for A615 reinforcing steel.

a,c

A Question of Ethics

Standing Up and Pushing Back

Among the technical sessions that formed part of the 141st Annual Civil Engineering Conference, which was held in Memphis, Tennessee, in October, was a one-hour seminar entitled Ethical Engineering Situations in Sustainability. Led by Steve Starrett, Ph.D., P.E., D.WRE, F.ASCE, an associate professor of civil engineering at Kansas State University, and featuring skits performed by students from the University of Memphis and the United States Military Academy at West Point, the seminar illustrated the sometimes difficult balance between an engineer's duties to a client and his or her obligation to serve the public interest and preserve natural resources. This article is based on one of those skits.

SITUATION: Barton Springs Pool, in Austin, Texas, is a man-made pool encompassing approximately 3 acres. Fed by underground springs and having an average year-round water temperature of 68°F, the pool has for many decades served as a popular swimming and recreational site for local residents and tourists alike. The pool and its surrounding areas are also the sole habitat of the Barton Springs salamander, a gilled salamander listed as an endangered species. As a result of the site's status as a protected habitat and as a recreational site, the City of Austin has adopted strict regulations to ensure that upstream development does not pollute the water and threaten the habitat.

In the situation portrayed during the seminar, a real estate developer has purchased a plot of land in the Barton Springs drainage basin and intends to use it to construct an apartment complex. The developer is seeking approval from the city health department to construct a septic system on the site and hires a local engineering firm to carry out percolation tests in support of the permit application. There is only a narrow window of availability in the area's construction season, and the developer is eager to obtain the permit so that ground can be broken.

An ASCE member and hydraulic engineer is assigned the task of collecting soil samples and running the percolation tests. However, of the four tests he conducts, only two samples show sufficient percolation rates, and he believes the city health department will not approve a permit based on these results. The engineer shares his results with his manager, who communicates the problem to the developer.

In response, the developer states that the septic system is essential to the project. He points out that the two unfavorable test results could be in error and suggests that the firm submit only the two results that buttress his permit application. He also emphasizes that he has long been a client of the engineering firm and that he expects the new development to generate much more work for the firm in the future.

The manager calls the engineer into her office and grills him about the test results, seeking to determine whether the unfavorable results could be faulty. The engineer states that he double-checked his results and found nothing "unusual" to suggest an error in the data. He points out that, if the soil doesn't percolate sufficiently, runoff from the drainage field could end up in Barton Springs Creek, thereby contaminating the pool and its surrounding areas. The member states that it is equally possible that the "good" results are erroneous and that the only way to be sure of the results is to conduct additional tests.

The manager insists that the firm has never missed its deadlines and that the developer is not a client she wishes to alienate. She states that the engineer needs to "make it work" with the data he has and that if a few results need to be scrapped because of possible errors, then that is what he should do.

QUESTION: Would the hydraulic engineer's submission of only favorable percolation test results to the city health department violate ASCE's Code of Ethics?

DISCUSSION: Canon 3 of the code is unambiguous: "Engineers shall issue public statements only in an objective and truthful manner." Category (b) in the guidelines to practice for this canon adds the following: "Engineers shall be objective and truthful in professional reports, statements, or testimony. They shall include all relevant and pertinent information in such reports, statements, or testimony."

By omitting the fact that two of his four tests indicated insufficient percolation, the engineer would clearly be excluding information that was relevant to the health department's decision on the permit.

By omitting the fact that two of his four tests indicated insufficient percolation, the engineer would clearly be excluding information that was relevant to the health department's decision on the permit. Moreover, his choice to exclude these results would not be based on an honest and objective belief that the data were faulty; rather, it would derive from his fear of the consequences he would face if the permit were denied. In submitting an incomplete and slanted report, the ASCE member would thus be violating his obligations under canon 3.

Canon 1 also is relevant: "Engineers shall hold paramount the safety, health, and welfare of the public…in the performance of their professional duties." If the engineer's submission of incomplete or untruthful test results would allow developed property to pollute the waters of Barton Springs, the engineer's actions might compromise public health and safety and thus would be in violation of canon 1.

Participants at the ASCE ethics seminar noted that while many clients can be demanding, the engineer had a greater obligation to ensure that the proposed development featured a sound wastewater disposal system with an acceptable effect on the environment. Many were of the opinion that the developer and the manager in this case "had crossed the line" in pushing the engineer to weed out unfavorable results and that it was unacceptable for the engineer to submit what amounted to a falsified report.

Although the question of the member's ethical obligations in this case might seem fairly straightforward, those attending the seminar noted that situations of this type can and do crop up in a member's professional life and that it might be difficult in practice to resist the pressure to exclude unfavorable data or invalidate an unfavorable result. Some attendees described situations in which they too had felt pressure from supervisors and peers to accommodate a valued client.

Others noted that it can take a lifetime to build a professional reputation but only one poor choice to destroy one. One member observed that, over the course of his lengthy career in the profession, the decisions he looked back on with the greatest sense of pride were those in which he had refused to take shortcuts.

ACKNOWLEDGMENTS: The skit featured at the seminar was prepared by Steve Starrett, Ph.D., P.E., D.WRE, F.ASCE, Carlos E. Bertha, Ph.D., James S. Talian, P.E., M.ASCE, Rebecca Waldrup, P.E., M.ASCE, Roger W. Meier, M.ASCE, Peter A. Sheydayi, P.E., D.WRE, M.ASCE, and David J. Prusak, P.E., D.WRE, M.ASCE. Special thanks are in order to Steve Starrett for his valuable suggestions for this column.

Participants at the ASCE ethics seminar noted that while many clients can be demanding, the engineer had a greater obligation to ensure that the proposed development featured a sound wastewater disposal system.

Members who have an ethics question or would like to file a complaint with the Committee on Professional Conduct may call ASCE's hotline at (703) 295-6061 or (800) 548-ASCE (2723), extension 6061. The attorneys staffing this line can provide advice on how to handle an ethics issue or file a complaint. Please note that individual facts and circumstances vary from case to case and that the general summary information contained in these case studies is not to be construed as a precedent binding upon the Society.

Hoke

Tara Hoke is ASCE's assistant general counsel and a contributing editor to Civil Engineering.

Document 2: Memo from Office Manager Establishing Panel

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D.C. 20555-0001

September 20, 2012

MEMORANDUM TO: Edwin Hackett, Chair
Bahqwat Jain, Member
Gordon Bjorkman, Member

FROM: Glenn M. Tracy, Director
Office of New Reactors

SUBJECT: AD HOC REVIEW PANEL - DIFFERING PROFESSIONAL OPINION ON THE AP1000 SHIELD BUILDING (DPO-2012-002)

In accordance with Management Directive (MD) 10.159, "The NRC Differing Professional Opinions Program," I am appointing you as members of a Differing Professional Opinion (DPO) Ad Hoc Review Panel (DPO Panel) to review a DPO that was forwarded to me to disposition.

This memorandum supersedes my August 29, 2012, memorandum. Bhaqwat Jain is replacing David Jeng as a member of the DPO panel because of Mr. Jeng's impending retirement.

The DPO (Enclosure 1) raises concerns related the AP1000 shield building which was reviewed and approved by the staff and certified through Commission rulemaking.

I have designated Edwin Hackett chairman of this DPO Panel and Gordon Bjorkman as a DPO Panel member. Bhaqwat Jain was proposed by the DPO submitter and serves as the third member of the DPO Panel. Panel membership has been discussed with the DPO submitter and he has no objection. In accordance with the guidance included in MD 10.159 and consistent with the DPO Program objectives, I task the DPO Panel to do the following:

- ❑ Review the DPO submittal to determine if sufficient information has been provided to undertake a detailed review of the technical issues.
- ❑ Meet with the submitter, as soon as practicable, to ensure that the DPO Panel understands the submitter's concerns and scope of the issues. (Normally within 7 days.)
- ❑ Promptly after the meeting, document the DPO Panel's understanding of the submitter's concerns, provide the Statement of Concerns (SOC) to the submitter, and request that the submitter review and provide comments, if necessary. (Normally within 7 days.)
- ❑ Maintain the scope of the review within those issues defined in the original written DPO and confirmed in the SOC.
- ❑ Consult with me as necessary to discuss schedule-related issues, the need for technical support (if necessary), or the need for administrative support for the DPO Panel's activities.
- ❑ Perform a detailed review of the technical issues and conduct any record review interviews, and discussions you deem necessary for a complete, objective, independent,

and impartial review. In particular, since this concern relates to a Commission-approved regulation (i.e. the AP100 Design Certification, Title 10 of the *Code of Federal Regulations* (10 CFR), Part 52 Appendix D), the panel should evaluate the technical merits of the DPO to determine if there is a sufficient basis to initiate the rulemaking process to modify 10 CFR 52 Appendix D. The panel should use the criteria in 10 CFR 52.63 "Finality of standard design certifications" in considering the need for a change to the AP1000 Design Certification.

If the panel recommends that a change to the AP1000 Design Certification should be pursued through rulemaking, the panel should then consider what, if any, actions should be taken on the AP1000 COL holders (Vogtle and Summer). Specifically, the panel should determine if the Vogtle and Summer site-specific Safe Shutdown Earthquakes (i.e. each specific design basis) would also raise concerns requiring regulatory action. The panel should evaluate the site-specific concerns using the criteria in 10 CFR 52.98 "Finality of combined licenses; information requests" in determining the need for site-specific backfits.

In addition, since the Commission issued the Aircraft Impact Assessment rule (10 CFR 150) as a beyond-design-basis requirement (as opposed to an "adequate protection" rule), the "adequate protection" basis for a proposed backfit does not appear relevant with respect to the aircraft impact protection concerns. To the extent the panel finds that 10 CFR 50.150 was not complied with the panel should consider which of the backfit criteria found in 10 CFR 52.63 and 10 CFR 52.98 should apply. All of the elements of 10 CFR 52.63 and 10 CFR 52.98 appear relevant to the seismic safety concerns.

The DPO Panel should re-interview individuals as necessary to clarify information during the review. In particular, the DPO Panel should have periodic discussions with the submitter to provide the submitter the opportunity to further clarify the submitter's views and to facilitate the exchange of information.

- ❑ Provide monthly status updates on your activities via email to Renée Pedersen, Differing Views Program Manager (DVPM) at the end of each month. This information will be reflected in the Milestones and Timeliness Goals for this DPO. Please provide a copy of email status updates to the submitter and to me.

- ❑ Issue a DPO Panel report, including conclusions and recommendations to me regarding the disposition of the issues presented in the DPO. The report should be a collaborative product and include all DPO Panel member's concurrence. Follow the specific processing instructions for DPO documents.

- ❑ Consult me as soon as you believe that a schedule extension is necessary to disposition the DPO.

- ❑ Recommend whether the DPO submitter should be recognized if the submitter's actions result in significant contributions to the mission of the agency.

Disposition of this DPO should be considered an important and time sensitive activity. The timeliness goal included in the MD for issuing a DPO Decision is 120 calendar days from the day the DPO is accepted for review. The timeliness goal for issuing this DPO Decision is November 13, 2012.

Process Milestones and Timeliness Goals for this DPO are included as Enclosure 2.

The timeframes for completing process milestones are identified strictly as goals - a way of working towards reaching the DPO timeliness goal of 120 calendar days. The timeliness goal identified for your DPO task is 70 calendar days.

Although timeliness is an important DPO Program objective, the DPO Program also sets out to ensure that issues receive a thorough and independent review. The overall timeliness goal should be based on the significance and complexity of the issues and the priority of other agency work. Therefore, if you determine that your activity will result in the need for an extension beyond the overall 120-day timeliness goal, please send me an email with the reason for the extension request and a new completion date. I will subsequently forward this request to the DVPM who will forward it to the EDO for approval.

Please ensure that all DPO-related activities are charged to Activity Code ZG0007.

Because this process is not routine, the DVPM will be meeting and communicating with all parties during the process to ensure that everyone understands the process, goals, and responsibilities. The DVPM will send you information intended to aid you in implementing the DPO process.

An important aspect of our internal safety culture includes respect for differing views. As such, you should exercise discretion and treat this matter sensitively. Documents should be distributed on an as-needed basis. In an effort to preserve privacy, minimize the effect on the work unit, and keep the focus on the issues, you should simply refer to the employee as the DPO submitter. Avoid conversations that could be perceived as "hallway talk" on the issue. We need to do everything that we can in order to create an organizational climate that does not chill employees from raising dissenting views.

As a final administrative note, please ensure that all correspondence associated with this case include the DPO number (DPO-2012-002) in the subject line, be profiled in accordance with ADAMS template OE-011, be identified as non-public and declared an official agency record when the correspondence is issued. Please email the ADAMS accession number for the record to DPOPM.Resource@nrc.gov and the record will be filed in the applicable DPO case file folder (DPO-2012-002) in the ADAMS Main Library. Following this process will ensure that a complete agency record is generated for the disposition of this DPO. If the submitter requests that the documents included in the DPO Case File be made public when the process is complete, you will be provided specific guidance to support a releasability review.

As a final administrative note, please include the DPO number (DPO-2012-002) in the subject line for all correspondence and **DO NOT place records associated with this case in ADAMS**. I have specifically requested the DVPM verify that the information within this DPO does not contain any proprietary information requiring specific controls. The results of this verification will be provided to the panel by the DVPM along with any appropriate actions. Distribute hard copies to those identified on correspondence and email a pdf of the signed original to those identified on distribution and to DPOPM.Resource@nrc.gov. Documents will be consolidated in one record, "DPO Case File" and placed into ADAMS when the DPO process is complete. If the submitter requests that the DPO Case File be made available to the public when the process is complete, I will ask you to help support a releasability review.

I appreciate your willingness to serve and your dedication to completing an independent and objective review of this DPO. Successful resolution of the issues is important for the NRC and its stakeholders. If you have any questions, you may contact me, Joe Williams, NRO Open, Cooperative Work Environment Champion, or Renée Pedersen, DVPM, at (301) 415-2742 or email Renee.Pedersen@nrc.gov.

I look forward to receiving your independent review results and recommendations.

Enclosures:
1. DPO-2012-002 (via hardcopy)
2. Milestones and Timeliness Goals

cc w/o Enclosure:
Submitter
DVPM

DPO Milestones and Timeliness Goals

DPO-2012-002: Acceptability of AP1000 Shield Building

Assigned to: Glenn Tracy, NRO

DPO Panel: Edwin Hackett, Chair; David Jeng, Member; Gordon Bjorkman, Member

DPO Milestone	**Timeliness Goals***	**Actual Date**
Individual submits DPO (NRC Form 680)	None	7/5/2012
DPOPM receives, screens, and accepts DPO	8 days	7/16/2012
DPOPM forwards DPO to office manager	7 days	7/18/2012
Office manager establishes DPO Panel	14 days	8/29/2012** 9/20/2012***
DPO Panel conducts review and issues report – meets with submitter (≈7 days) – establishes Statement of Concern (≈7 days) – confirms schedule with office manager (≈7 days) – completes review (≈ 49 days after start of review) – writes report (≈21 days after completion of review)	70 days	
Office manager issues DPO Decision	21 days	
DPO TIMELINESS (time from acceptance of DPO to DPO Decision)	120 days 11/13/ 2012	

*The timeframes for completing process milestones are identified strictly as goals—a way of working towards reaching the Differing Professional Opinions (DPO) timeliness goal of 120 calendar days.

The timeliness goal for dispositioning a DPO (i.e., DPO Decision) will be established as 120 calendar days after a DPO has been accepted for review under the DPO Program.

Office managers should send requests for extension beyond the 120-day timeframe to the Differing Professional Opinions Program Manager (DPOPM), who will forward the request to the Executive Director for Operations with a recommendation.

**Additional time was required to identify qualified panel members agreeable to the submitter.
***New panel member designated due to pending retirement in coordination with submitter.

Enclosure 2

Document 3: DPO Panel Report

February 24, 2014

MEMORANDUM TO: Glenn Tracy, Director
Office of New Reactors

FROM: E.M. Hackett, DPO Panel Chair
[redacted], DPO Panel Member
[redacted] DPO Panel Member

SUBJECT: DIFFERING PROFESSIONAL OPINION PANEL REPORT ON DPO-2012-002 STRUCTURAL INTEGRITY CONCERNS WITH THE AP1000 SHIELD BUILDING

In a memorandum dated September 20, 2012, you appointed us as members of a Differing Professional Opinion (DPO) Ad Hoc Review Panel (DPO Panel) to review a DPO regarding structural integrity concerns with the AP1000 shield building. The DPO Panel has reviewed the DPO in accordance with the guidance in Management Directive 10.159, The NRC Differing Professional Opinions Program.

The results of the DPO Panel's evaluation of the concerns raised in the DPO are detailed in the enclosed DPO Panel Report. The DPO contained four safety concerns that were outlined in the Statement of Concerns agreed to by the DPO Submitter. These concerns fall into two broad categories: design basis demand and capacities (e.g., seismic, strength and ductility) and beyond design-basis demand and capacities (Aircraft Impact Assessment - AIA). The Panel retained two technical experts to assist in the review of the safety concerns. Based on a thorough technical review and the collective expertise and judgment of the Panel and the experts, the DPO Panel concluded that:

1. Safety Concern # 1 - The certified design of the AP 1000 shield building does not meet the NRC's seismic margin requirements

   The Panel concludes that the AP1000 SB does meet the NRC's seismic margin requirements.

   The Panel also concluded that: (a) The Westinghouse Electric Company (WEC) did not calculate the High Confidence Low Probability of Failure (HCLPF) values consistent with the shield building reanalysis results that were reported in Appendix L to the shield *building design report, and recommends that the staff follow-up on the revised HCLPFs for shield building structures, and; (b) the HCLPF values reported in Table 19.55-1 of the DCD are at variance with the ones reported in Chapter 11 (pages 16-23) of the shield building design report, and recommends that the staff follow-up to resolve this discrepancy (Section 4.1.3).*

2. Safety Concern # 2 - The NRC staff's conclusion that the aircraft missile would not penetrate the AP1000 shield building wall is not logical.

   The Panel concludes that the AP1000 Shield Building shell, constructed of steel-concrete (SC) modules, would not be perforated by the aircraft impact at room temperature (Section 4.2.3). This conclusion is based on three independent AIAs performed by WEC, the NRC Office of Research (RES), and the Panel's expert, Dr. Rashid. All three concluded that the AP1000 SB shell constructed of steel-concrete (SC) modules was not perforated by the aircraft impact at room temperature.

   The Panel's conclusions on temperature effects for the Shield Building AIA, the AIA for the Passive Cooling System (PCS) tank and benchmarking of computer models for AIA, are provided below:

   (a) Temperature effects in the AIA evaluation of the Shield Building Wall – Although this issue was not specifically raised in the SOC, the Panel considered it important to address the issue. The Panel members were of different opinions on this issue:

   Panel members [redacted] and Hackett consider that the WEC evaluation of this issue was appropriate and in accordance with the NEI AIA guidance regarding use of best-estimate properties for the beyond design basis assessment (Section 4.2.3.5.1(a)).

   Panel member [redacted] agrees with the Submitter's concern stated as, "the ability of the shield wall to resist aircraft impact under cold weather has not been substantiated and is doubtful." He recommends that the staff follow-up on this issue (Section 4.2.3.5.1(b)).

   (b) The AIA for the PCS tank - Based on the evaluation in Section 4.2.4 of the report, the Panel understands that the structural integrity of the PCS tank and steel containment to withstand physical and shock effects due to aircraft impact was not inspected and verified by the NRC staff. Although the assessment was reviewed by the ACRS, the Committee did not conduct a quantitative verification of the WEC analyses. The Panel was informed by WEC regarding an overview of the AIA for the PCS tank that they performed, but we could not perform our own independent assessment due to insufficient resources. Therefore, the Panel concludes that that it cannot provide an independent opinion on the reasonable assurance of the validity of the WEC assessment of the PCS tank impact and the steel containment assessment for

potential debris impacts. The Panel recommends that the staff follow-up on this issue and perform a review of the WEC assessments of these critical structures (Section 4.2.4).

(c) Benchmarking of computer models for AIA - the Panel concludes that it is not able to determine that WEC has met the intent of NEI 07-13 subsection 2.4.1(4) with regards to benchmarking of computer models for AIA.

3. Safety Concern # 3 - The certified shield building wall does not possess sufficient strength and ductility to resist an earthquake and/or aircraft missile impact loading that is specified by the NRC (Section 4.3).

   The Panel concludes that he AP1000 SB possesses both sufficient strength and ductility to be appropriately resistant to seismic excitation and aircraft impact per NRC requirements. In particular, the Panel finds that:

(a) The Submitter's concern regarding additional demands due to "the irregular (Zigzag)" boundary condition, accidental torsion, and the P-Delta effects either have already been considered or do not result in higher design demands (Section 4.3.2 (a)).

(b) With regard to the Submitter's concern regarding the capacity of the Steel-Concrete (SC) module based on ACI Code 349, the Panel finds that WEC appropriately considered the pertinent ACI Code procedures, criteria and requirements. The Panel also finds that many of the ACI Code equations are applicable to the SC module element used in the AP1000 design (Section 4.3.2 (b) i and ii).

(c) The Panel agrees with the Submitter that the ACI Code requires that ductility should be in every single structural component of the structure and that all members of the lateral force resisting system should be detailed for seismic ductile performance. (Section 4.3.2 (b) iii).

(d) The shield building SC modules as designed, will behave in a ductile manner even though some SC module elements with low shear-span-to-depth ratios failed in a brittle manner in laboratory tests. The Panel notes that RC beams with low shear-span-to-depth ratios also fail in a brittle manner (Section 4.3.2 (b) iv and Section 4.2.3.1).

(e) The Panel does not agree with the Submitter's concern regarding the use of a non-linear static push over analysis in lieu of the ACI code requirements. The Panel finds that the design basis of the shield building for the safe shutdown earthquake (SSE) is based on linear elastic response analysis and not on a non-linear static push over analysis. The push over analysis provided a qualitative understanding of the beyond design basis response and did not replace the ACI 349 design procedure or NRC

requirements for the SSE. The use of static push over analysis for purposes other than the design is considered appropriate (Section 4.3.2 (b) v).

(f) The Panel agrees with the Submitter's concern that WEC did not consider a reduction in the ACI code capacity for in-plane shear strength due to interaction with out-of-plane shear forces. However, we conclude that this reduction in the in-plane shear strength is negligible and is not required by the ACI 349 Code (Section 4.3.2 (b) vi).

4. Safety Concern # 4 - The Certified shield building design does not meet General Design Criteria (GDC) 2 requirements.

   On the basis of our conclusions regarding Safety concerns # 1 and 3 above, the Panel concludes that the AP1000 SB meets the requirements of General Design Criterion 2, "Design bases for protection against natural phenomena." (Section 4.4.2)

During the course of our review, the Panel made certain observations regarding the design, analysis and construction aspects of the shield building and the staff's technical and regulatory review processes. The Panel identified certain issues for the staff to follow-up. The observations and associated recommendations are detailed in Section 7.0 of the report. One of the observations deals with the shear reinforcing details at the connection of the PCS tank outer wall to the conical roof (Section 7.4.4). The Panel strongly recommends that this detail be modified. Another observation deals with the design and constructability of the tapered SC shield wall in the air-inlet region transitioning from 54 inch to 36 inch in thickness (Section 7.1). The Panel strongly recommends that the shield wall transition detail in the air inlet region be reviewed to ensure the tapered region is adequately designed, detailed, and is constructible. The Panel also recommended that the staff follow up on the design and analysis of the PCS tank for design basis loads (Sections 7.4.1, 7.4.2, 7.4.3 and 7.4.5).

Prior to completion of the Panel's review, one of the experts retained by the Panel, Mr. Loring Wyllie, transmitted a letter to the Panel [a] highlighting and emphasizing two concerns he had previously identified in his report [b]. Due to the added emphasis, tone and contents of the letter, the Panel specifically referred these concerns to NRO via email on December 28, 2012 [c] for appropriate disposition.

Finally, the Panel made some observations and recommendations regarding the DPO process and will forward these under separate cover to the DPO Project Manager.

References

[a] "AP1000 Shield Building," Letter from Loring Wyllie, Jr. to Panel Member [redacted] December 13,

[b] "Expert Report of Selected Questions, AP1000 Enhanced Shield Building," Loring A. Wyllie, Jr., June, 2013

[c] Email from Panel Chair E. Hackett, to NRO Director, G. Tracy, December 27, 2013

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E.M. Hackett, ACRS

Differing Professional Opinion (DPO) Regarding Structural Integrity Concerns with the AP1000 Shield Building (DPO-2012-002)

DPO Panel Report

E.M. Hackett, Ph.D.

Name, Panel Chair

[redacted]

Name, Panel Member

[redacted]

Name, Panel Member

Date: February 24, 2014

Acknowledgements

The Panel appreciates the support of NRO throughout the DPO process. The NRO Office Director, Glenn Tracy, maintained a strong commitment to supporting a thorough, objective and independent evaluation of the Submitter's concerns. He assured that the Panel had the resources necessary to carry out the tasking. The Panel also recognizes the outstanding efforts of George Wunder of NRO, as Project Manager. In addition, the Panel appreciates the efforts of NRO intern Alex Burja in assisting with the evaluation and categorization of numerous email communications pertinent to the DPO.

Joe Williams and Jim Steckel of NRO, as the Office OCWE Champions, facilitated communications with the Submitter and provided continuity throughout the process. Bern Stapleton of NSIR provided expert assistance regarding the assessment of the sensitivity of information and communications pertinent to the DPO. Sara Kirkwood of OGC provided guidance regarding protocols for contacts with the Westinghouse Electric Corporation (WEC). Dan Dorman of NRR provided a detailed review of the final draft of the report for consistency and completeness.

The Panel retained the services of two expert consultants to assist us with our evaluation of the DPO concerns. We hereby acknowledge the outstanding technical support provided by Dr. Joe Rashid and Mr. Loring Wyllie. Alesha Bellinger of ACRS and [redacted] of NRO and her staff were instrumental in the development and execution of the contracts for Dr. Rashid and Mr. Wyllie. Sherry Meador of ACRS formatted the final draft of the report and profiled and entered the report and several references in ADAMS.

As the owners of the DPO process, OE provided expert guidance throughout our efforts. In this regard, we would like to particularly acknowledge the contributions of Renee Pedersen, Marge Sewell and Dave Solorio of OE.

List of Abbreviations and Acronyms

AASHTO	American Association of State Highway and Transportation Officials
AB	Auxiliary Building for AP1000
ABAQUS	Finite Element Structural Evaluation Code
ACI	American Concrete Institute
ACRS	Advisory Committee on Reactor Safeguards
ABWR	Advanced Boiling Water Reactor
AIA	Aircraft Impact Assessment
AISC	American Institute of Steel Construction
ANSYS	Finite Element Structural Evaluation Code
AP1000	WEC Advanced Passive Pressurized Water Reactor
CIS	Containment Internal Structures
CSDRS	Consolidated Seismic Design Response Spectra
CVN	Charpy V-Notch impact test
DCD	Design Certification Document
DPO	Differing Professional Opinion
ESBWR	Economic Simplified Boiling Water Reactor
FEMA	Federal Emergency Management Agency
FSER	Final Safety Evaluation Report
HCLPF	High Confidence of Low Probability of Failure
LSDYNA	Finite Element Structural Evaluation Code
NEHRP	National Earthquake Hazards Reduction Program
NEI	Nuclear Energy Institute
NII	Nuclear Installations Inspectorate (UK)
NIST	National Institute for Standards and Technology
NDT	Nil-Ductility Transition Temperature
NRO	NRC Office of New Reactors
NRR	NRC Office of Nuclear Reactor Regulation
NSIR	NRC Office of Nuclear Security and Incident response
OCWE	Open Collaborative Work Environment
OE	NRC Office of Enforcement
OEDO	NRC Office of the Executive Director for Operations
OGC	NRC Office of the General Counsel
PCS	Passive Cooling System
RC	Reinforced Concrete
RES	NRC Office of Nuclear Regulatory Research
RG	Regulatory Guide
RLE	Review Level Earthquake
SB	Shield Building
SC	Steel-Concrete Composite
SCC	Self-Consolidating Concrete
SOC	Statement of Concerns
SRM	Staff Requirements Memorandum

SRP	Standard Review Plan
SRSS	Square Root Sum of Squares
SSE	Safe Shutdown Earthquake
STP	South Texas Project
STPNOC	South Texas Project Nuclear Operating Company
WEC	Westinghouse Electric Corporation

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Executive Summary

This report provides an independent and objective review of a differing professional opinion (DPO) regarding safety concerns with the structural integrity of the certified AP1000 shield building (SB) design. DPO-2012-002 was submitted on July 6, 2012 and is focused on four safety concerns with the structural integrity of the shield building. The DPO Panel was tasked to:

Perform a detailed review of the technical issues (as described in the Statement of Concerns (SOC) – Appendix A) and conduct any document reviews, interviews and discussions necessary for a complete, objective, independent and impartial review

The Panel also reviewed all email communications of technical information that the DPO submitter provided to the Panel for its consideration. Appendix D provides a listing of the emails and how they were addressed by the Panel. Appendix E provides a crosswalk between the DPO issues and specific emails. This report provides the Panel's conclusions and recommendations and the technical bases supporting the conclusions and recommendations.

The four safety concerns identified in the DPO fall into two broad categories: design basis demand and capacities (e.g., seismic, strength and ductility) and beyond design-basis demand and capacities (Aircraft Impact Assessment - AIA). The Panel retained two technical experts to assist in the review of the safety concerns. Based on a comprehensive technical review and the collective expertise and judgment of the Panel and the experts, the DPO Panel concluded that:

1. Safety Concern # 1 - The certified design of the AP 1000 shield building does not meet the NRC's seismic margin requirements

   The Panel concludes that the AP1000 SB does meet the NRC's seismic margin requirements.

   The Panel also concluded that: (a) The Westinghouse Electric Company (WEC) did not calculate the High Confidence Low Probability of Failure (HCLPF) values consistent with the shield building reanalysis results that were reported in Appendix L to the shield building design report, and recommends that the staff follow-up on the revised HCLPFs for shield building structures, and; (b) the HCLPF values reported in Table 19.55-1 of the DCD are at variance with the ones reported in Chapter 11 (pages 16-23) of the shield building design report, and recommends that the staff follow-up to resolve this discrepancy (Section 4.1.3).

2. Safety Concern # 2 - The NRC staff's conclusion that the aircraft missile would not penetrate the AP1000 shield building wall is not logical.

   The Panel concludes that the AP1000 Shield Building shell, constructed of steel-concrete (SC) modules, would not be perforated by the aircraft impact at room temperature (Section 4.2.3). This conclusion is based on three independent AIAs performed by WEC, the NRC Office of Research (RES), and the Panel's expert, Dr. Rashid. All three concluded that the AP1000 SB shell constructed of steel-concrete (SC) modules was not perforated by the aircraft impact at room temperature.

   The Panel's conclusions on temperature effects for the Shield Building AIA, the AIA for the Passive Cooling System (PCS) tank and benchmarking of computer models for AIA, are provided below:

   (a) Temperature effects in the AIA evaluation of the Shield Building Wall – Although this issue was not specifically raised in the SOC, the Panel considered it important to address the issue. The Panel members were of different opinions on this issue:

   Panel members [redacted] and Hackett consider that the WEC evaluation of this issue was appropriate and in accordance with the NEI AIA guidance regarding use of best-estimate properties for the beyond design basis assessment (Section 4.2.3.5.1(a)).

   Panel member [redacted], agrees with the Submitter's concern stated as, "the ability of the shield wall to resist aircraft impact under cold weather has not been substantiated and is doubtful." He recommends that the staff follow-up on this issue (Section 4.2.3.5.1(b)).

   (b) The AIA for the PCS tank - Based on the evaluation in Section 4.2.4 of the report, the Panel understands that the structural integrity of the PCS tank and steel containment to withstand physical and shock effects due to aircraft impact was not inspected and verified by the NRC staff. Although the assessment was reviewed by the ACRS, the Committee did not conduct a quantitative verification of the WEC analyses. The Panel was informed by WEC regarding an overview of the AIA for the PCS tank that they performed, but we could not perform our own independent assessment due to insufficient resources. Therefore, the Panel concludes that that it cannot provide an independent opinion on the reasonable assurance of the validity of the WEC assessment of the PCS tank impact and the steel containment assessment for potential debris impacts. The Panel recommends that the staff follow-up on this issue and perform a review of the WEC assessments of these critical structures (Section 4.2.4).

(c) Benchmarking of computer models for AIA - the Panel concludes that it is not able to determine that WEC has met the intent of NEI 07-13 subsection 2.4.1(4) with regards to benchmarking of computer models for AIA.

3. Safety Concern # 3 - The certified shield building wall does not possess sufficient strength and ductility to resist an earthquake and/or aircraft missile impact loading that is specified by the NRC (Section 4.3).

   The Panel concludes that he AP1000 SB possesses both sufficient strength and ductility to be appropriately resistant to seismic excitation and aircraft impact per NRC requirements. In particular, the Panel finds that:

   (a) The Submitter's concern regarding additional demands due to "the irregular (Zigzag)" boundary condition, accidental torsion, and the P-Delta effects either have already been considered or do not result in higher design demands (Section 4.3.2 (a)).

   (b) With regard to the Submitter's concern regarding the capacity of the Steel-Concrete (SC) module based on ACI Code 349, the Panel finds that WEC appropriately considered the pertinent ACI Code procedures, criteria and requirements. The Panel also finds that many of the ACI Code equations are applicable to the SC module element used in the AP1000 design (Section 4.3.2 (b) i and ii).

   (c) The Panel agrees with the Submitter that the ACI Code requires that ductility should be in every single structural component of the structure and that all members of the lateral force resisting system should be detailed for seismic ductile performance. (Section 4.3.2 (b) iii).

   (d) The shield building SC modules as designed, will behave in a ductile manner even though some SC module elements with low shear-span-to-depth ratios failed in a brittle manner in laboratory tests. The Panel notes that RC beams with low shear-span-to-depth ratios also fail in a brittle manner (Section 4.3.2 (b) iv and Section 4.2.3.1).

   (e) The Panel does not agree with the Submitter's concern regarding the use of a non-linear static push over analysis in lieu of the ACI code requirements. The Panel finds that the design basis of the shield building for the safe shutdown earthquake (SSE) is based on linear elastic response analysis and not on a non-linear static push over analysis. The push over analysis provided a qualitative understanding of the beyond design basis response and did not replace the ACI 349 design procedure or NRC requirements for the SSE. The use of static push over analysis for purposes other than the design is considered appropriate (Section 4.3.2 (b) v).

(f) *The Panel agrees with the Submitter's concern that WEC did not consider a reduction in the ACI code capacity for in-plane shear strength due to interaction with out-of-plane shear forces. However, we conclude that this reduction in the in-plane shear strength is negligible and is not required by the ACI 349 Code (Section 4.3.2 (b) vi).*

4. Safety Concern # 4 - The Certified shield building design does not meet General Design Criteria (GDC) 2 requirements.

   On the basis of our conclusions regarding Safety concerns # 1 and 3 above, the Panel concludes that the AP1000 SB meets the requirements of General Design Criterion 2, "Design bases for protection against natural phenomena." (Section 4.4.2)

During the course of our review, the Panel made certain observations regarding the design, analysis and construction aspects of the shield building and the staff's technical and regulatory review processes. The Panel identified certain issues for the staff to follow-up. The observations and associated recommendations are detailed in Section 7.0 of the report. One of the observations deals with the shear reinforcing details at the connection of the PCS tank outer wall to the conical roof (Section 7.4.4). The Panel strongly recommends that this detail be modified. Another observation deals with the design and constructability of the tapered SC shield wall in the air-inlet region transitioning from 54 inch to 36 inch in thickness (Section 7.1). The Panel strongly recommends that the shield wall transition detail in the air inlet region be reviewed to ensure the tapered region is adequately designed, detailed, and is constructible. The Panel also recommended that the staff follow up on the design and analysis of the PCS tank for design basis loads (Sections 7.4.1, 7.4.2, 7.4.3 and 7.4.5).

Prior to completion of the Panel's review, one of the experts retained by the Panel, Mr. Loring Wyllie, transmitted a letter to the Panel [a] highlighting and emphasizing two concerns he had previously identified in his report [b]. Due to the added emphasis, tone and contents of the letter, the Panel specifically referred these concerns to NRO via email on December 28, 2012 [c] for appropriate disposition.

Finally, the Panel made some observations and recommendations regarding the DPO process and will forward these under separate cover to the DPO Project Manager.

References

[a] "AP1000 Shield Building," Letter from Loring Wyllie, Jr. to Panel Member [redacted] December 13, 2013

[b] "Expert Report of Selected Questions, AP1000 Enhanced Shield Building," Loring A. Wyllie, Jr., June, 2013

[c] Email from Panel Chair E. Hackett to NRO Director G. Tracy, December 27, 2013

1.0 Introduction

DPO-2012-002 was formally submitted on July 6, 2012 [1(a)]. Broadly, the DPO is focused on concerns with the structural integrity of the AP1000 shield building. More specifically, these concerns fall into two categories: (1) design basis loadings (seismic, strength and ductility) and; (2) beyond design-basis loading (AIA). The DPO Panel was established via memorandum from the NRO Office Director, dated September 20, 2012 [1(b)]. The DPO Panel was tasked to:

(a) Review the DPO submittal to determine if sufficient information has been provided to undertake a detailed review of the technical issues.
(b) Meet with the Submitter as soon as practicable to ensure that the DPO Panel understands the Submitters concerns and scope of the issues
(c) Document the DPO Panel's understanding of the Submitters concerns via development of a Statement of Concerns (SOC) and request the Submitter's comments on the SOC
(d) Maintain the scope of the review within the original DPO and SOC
(e) Consult with the NRO Office Director (OD) as needed regarding schedule issues, the need for technical support or the need for administrative support
(f) Perform a detailed review of the technical issues and conduct any interviews and discussions necessary for a complete, objective, independent and impartial review
(g) Provide periodic status updates
(h) Issue a DPO Panel Report
(i) Consult the NRO OD regarding the need for any schedule extensions
(j) Recommend whether the DPO Submitter should be recognized if the Submitters actions result in significant contributions to the mission of the agency

With regard to (b) and (c), the Panel met with the Submitter on October 18 and October 25, 2012, to discuss the SOC. The Panel continued to iterate development of the SOC via Panel meetings and email communications with the Submitter during November, 2012 and obtained the Submitter's concurrence on the SOC on November 29, 2012 [Appendix A].

With regard to (e), due to the nature and complexity of the issues raised by the Submitter, the Panel considered that additional technical support would be required. This was communicated to the NRO Office Director in October, 2012. The NRO OD subsequently approved use of NRO contract resources to provide the Panel with additional technical support. In addition, the NRO OD directed a senior NRO PM to provide project management support for the DPO Panel. The Panel greatly appreciates the additional resources provided by NRO for a thorough evaluation of the Submitter's concerns.

The detailed nature of the concerns also led the Panel to require discussions with the Westinghouse Electric Corporation (WEC), the vendor for the AP1000. WEC was very cooperative in providing additional information to the Panel during several teleconference discussions. OGC and NSIR were helpful in guiding the Panel regarding protocols for these discussions with WEC due to the sensitive nature of the DPO process and protection of security-related information.

With regard to (i), the nature and complexity of the Submitter's concerns, combined with the need for contract assistance and contacts with WEC led to the need to extend the schedule significantly beyond the "typical" expectations for processing a DPO. Schedule extensions were processed in accordance with agency DPO guidance and approved by OEDO.

As already alluded to above, the DPO Panel performed its review by: interviewing the DPO Submitter, developing an SOC and obtaining the Submitter's concurrence on the SOC, conducting document reviews, interviewing members of the staff and other stakeholders, interviewing the applicant's staff, and obtaining contract assistance from outside technical experts. The Panel also reviewed all email communications of technical information that the DPO submitter provided to the Panel for its consideration (Appendices D and E). As part of this review, the Panel did not consider and respond to all technical issues (brought forward by the submitter subsequent to SOC) that the Panel considered were not within its scope or were not part of the SOC.

It is important to recognize the limitations inherent to this, or any DPO Panel's activities. Due to schedule and resource constraints, the Panel was limited in its ability to pursue certain issues or to conduct independent detailed verifications of all aspects of the concerns. Rather, the Panel has endeavored to provide the best possible assessment given the constraints imposed. Such an assessment necessarily relies on expert judgments.

1.1 References

1(a) DPO-2012-002, "The Certified AP100 Shield Building is Unsafe," NRC Form 680, July 6, 2012

1(b) Memorandum to Edwin Hackett, [redacted] and [redacted] from Glenn Tracy, "AD HOC REVIEW PANEL - DIFFERING PROFESSIONAL OPINION ON THE AP1000 SHIELD BUILDING (DPO-2012-002), September 20, 2012

2.0 Background

2.1 Evolution of the AP1000 Shield Building Design

The AP1000 shield building design as described in the AP1000 DCD, Revision 15, was a reinforced concrete (RC) design. In AP1000 Revisions 16 and 17, a new shield building design was proposed. The revised design incorporates a steel-concrete (SC) composite structure as opposed to the traditional RC nuclear shield building construction.

The following is a description of the features of the revised design which is taken in large part from the ACRS Letter Report on the matter, dated December 13, 2010 [2(a)]:

The key features of the revised shield building are: a cylindrical wall which comprises the bulk of the structure constructed of SC modules; a conical RC roof structure with an integral RC water tank which contains approximately 7 million pounds of water; a tension ring at the intersection of the roof with the cylindrical wall consisting of a built-up closed section of steel plates filled with concrete; and mechanical connections that join the SC wall to the basemat and the RC wall of the auxiliary building.

The tension ring is designed as a steel structure in accordance with the American National Standards Institute/American Institute of Steel Construction (ANSI/AISC) N690. The steel frame for the roof is designed to the applicable building code, ANSI/AISC N690. The concrete roof is designed to American Concrete Institute (ACI) 349 requirements without credit for the steel plate on the bottom of the concrete. The SC modules have not been used previously in nuclear construction in the United States.

In the initial design proposed for the new shield building, the SC wall module for the 3-foot thick cylindrical wall consisted of steel faceplates with attached [ ] a,c long steel studs which were embedded in the 35-inch thick concrete fill between the two plates. In a letter dated October 15, 2009 [2(b)], the NRC staff determined that this design would require modifications to ensure its ability to perform its safety function under design basis loading conditions. Some key issues identified in the letter are listed below:

- The need to demonstrate the adequacy of the design and detailing of the SC module to function as a fully composite unit, as assumed in the WEC design and analysis.

- The need to demonstrate the adequacy of the design and detailing of the connection between the SC module wall and RC wall of the auxiliary building to withstand all design basis loads.

- The need to support the design and analysis of the shield building tension ring (i.e., ring girder) and the air-inlet region with a validated analysis method (i.e., benchmarked to experimental data) or by confirmatory model tests.

During the review, NRC staff concerns focused particularly on the lack of transverse reinforcement that would tie one faceplate to the opposite faceplate to ensure that the SC modules would function as a unit for either out-of-plane demands or in-plane demands. The DPO Submitter played a key role in articulating the staff's concerns in this area.

In response to the staff's concerns, WEC developed a revised design for the shield building that added tie bars welded to opposite faceplates in the SC wall modules, and also revised the design of the ring girder and the connections between the SC wall module and the RC wall. The revised SC wall module has thicker faceplates, as well as tie bars between the plates to help ensure that the module acts as a composite unit with increased out-of-plane shear strength. The spacing between the tie bars is greater in regions of the wall away from discontinuities and connections, which have low out-of-plane demands, than it is in the regions near discontinuities and the SC to RC connections, where out-of-plane shear demands are higher.

Although design codes for SC modular construction for some applications have been developed in Japan, codes and standards for the design of SC structural components do not exist in the United States. WEC used ACI-349, a design code for RC in nuclear safety-related structures, to guide their design of the SC cylindrical wall modules. Even though the scope of ACI-349 does not include SC construction, the underlying design philosophy, elastic behavior and strength for design basis loads and resilience through ductility for beyond design-basis loads, does apply. Also, the underlying assumptions on composite behavior of steel and concrete materials in RC structural elements do apply to SC structural elements.

To validate this adaptation of ACI-349, WEC conducted a testing program at Purdue University. The tests were intended: (1) to demonstrate that the adaptations of ACI-349 proposed by WEC could be used to predict the out-of-plane shear strength, flexural capacity, and in-plane shear strength of SC structures and (2) to investigate the failure behavior of the SC modules.
The test results were also used to benchmark the finite element analyses performed to support the design of the shield building. WEC's approach to developing the design basis involved three levels of analysis with increasing levels of model refinement. Level 1 was used for determining the load magnitudes (seismic demands) imposed on the structure. It was a linear elastic analysis with a fairly coarse mesh that used simplified models to account for concrete cracking. Level 2 was also a linear elastic analysis with a more refined mesh used for determining the member forces and deformation demands. Level 3 was a nonlinear analysis used to assess the region with high stresses, strains, and displacements in the shield building, such as the connection regions. Detailed sub-models were used which included elements such as concrete, steel plates, studs, and tie bars. A strain-based failure criterion was selected to define acceptable limits under design-basis loads. The analysis models were benchmarked against the Purdue tests.

NRO subsequently requested that the Office of Nuclear Regulatory Research (RES) provide assistance in evaluating the structural analysis, design, construction, and inspection methods for the AP1000. The findings in the RES report [2(c)] were used to inform the evaluation of the shield building design by the staff of NRO. RES engaged outside recognized experts in the field of reinforced concrete structures and composite structures. These experts were not the same experts that were retained by the DPO panel to assist us in our evaluation. RES staff assessed and consolidated the inputs from each expert and performed their own independent assessment to develop their report. The Panel notes that the RES evaluation of the shield wall design report was based on Revision 2 (2010) of the enhanced shield building design report that was subsequently revised in 2011 (Revision 4) to address the design basis combination of both seismic and thermal effects, and is the basis for AP1000 design certification.

2.2 Conclusions of Previous Related Evaluations

The RES staff concluded that the agreement between the experimental results and the predictions of the Level 3 finite element models were adequate to benchmark the models for loads up to and beyond the design-basis safe shutdown earthquake (SSE). The RES staff also concluded that the models would provide useful predictions of SC module behavior for load levels beyond the design-basis level and below the self-imposed analysis strain limits.
The NRO staff concluded that WEC demonstrated that the models used for the analysis of the shield building predicted the observed experimental behavior and response with acceptable accuracy up to the design-basis SSE seismic load level. Also, the staff found that the design had acceptable stress and strain values in the SC steel plates, tie bars, and studs. The staff also found that WEC's adaptation of the ACI-349 Code for the design of the SC modules is acceptable. Finally, the staff found WEC's confirmatory analysis approach to be acceptable.

The test specimens representing the SC modules with the closer tie-bar spacing used in regions of high out-of-plane demands failed in a ductile manner in all the tests performed at Purdue University . Some of the test specimens representing the SC wall modules with the tie-bar spacing used in the regions of low out-of-plane shear demands failed in a non-ductile manner in out-of-plane shear tests. This non-ductile behavior was the primary basis for a non-concurrence [2(d)] advanced at that time by the DPO Submitter on the acceptability of the design of the shield building. In the view of the Submitter, the behavior of the modules with increased tie-bar spacing was unacceptable. This non-concurrence was reviewed extensively by both the staff [2(e)] and ACRS [2(a)]. In their report, the ACRS noted, as a matter of principle, that structures important to nuclear safety should be designed so that, in the unlikely event the loads acting on the structure are larger than anticipated, the structure would behave in a ductile manner.

The Submitter maintained that this principle should be met by every element of the structure. WEC contended that it is the structure as a whole, not its elements, that ultimately matters, and that the design of the shield building does provide a structure that will behave in a ductile manner, because the low-ductility elements will approach their elastic limits only after those elements of the structure that do behave in a ductile manner have undergone significant plastic deformation.

The ACRS concluded that the WEC approach was consistent with the intent of ACI-349, which requires ductile behavior only where demands are high and plastic deformation is expected to occur. [1] In the regions of low out-of-plane shear demands, the analysis shows that the out-of-plane shear capacity of the low ductility module is about 5 times greater than the applied shear load under design-basis loads. Indeed, except for some very small regions, the capacity is typically 10 times greater than the demand. Because the structural analysis follows typical seismic engineering practice and the finite element models used to describe the behavior of the SC models have been benchmarked to show satisfactory agreement with experiments even for loads greater than the design-basis loads, the NRO staff found this margin to be acceptable, despite the uncertainties associated with any seismic analysis. The ACRS concurred with the staff's conclusion and also found that this conclusion was consistent with the independent evaluation by the RES staff. All four of the consultants engaged by RES also agreed that the demand-to-capacity ratio was acceptable with sufficient margin. An additional expert consultant engaged by the ACRS, also agreed that margins were sufficient to ensure that the overall structural behavior was ductile. The ACRS also concluded that the effort and scope of analysis and assessment required for the shield building in this case suggests that if SC composites are to be more widely used in nuclear applications, a consensus code should be developed, as has been done for other types of nuclear construction.

With regard to assessments for aircraft impact (10 CFR 50.150), WEC performed an analysis demonstrating that the shield building would not be perforated by aircraft impact. The NRC staff concurred with this assessment based, in part, on an independent AIA performed by RES. As part of their evaluation, WEC did not initially consider an impact on the PCS tank, but subsequently performed an assessment in this area in response to questions from the ACRS. The NRC staff did not subsequently evaluate and perform a detailed inspection of this assessment. However, the ACRS did review the potential PCS tank impact as part of its evaluation of AIA for the AP1000 [2(g)]. Overall, the ACRS concluded that the design, "complies with the requirements of 10 CFR 50.150." The ACRS also recommended that the staff should evaluate information and analyses presented to the ACRS to determine if there is a need for further inspections. WEC informed the Panel that staff did not perform further inspections of the analyses presented to the ACRS. The Panel's evaluation for AIA of the design is contained in Section 4.2.

(1) The Panel and our expert, Mr. Wyllie, evaluated the ACI Code requirements in this regard in detail. The ACI code does specify that "ductility should be in every single structural component of the structure." However, Mr. Wyllie noted that since the AP1000 shield building is a shear wall system, the numerous detailed requirements in the code that apply to rebar detailing for ductile performance do not apply. A detailed discussion is provided in Section 4.3.2 (b) iv of our report.

2.3 DPO 2012-002

The Submitter considered that the conclusions from the reviews cited in Section 2.2 (above) did not satisfy the concerns that had been raised. The Submitter therefore subsequently decided to pursue further resolution through the DPO process. Accordingly, DPO-2012-002 was formally submitted on July 6, 2012 [2(f)]. In the DPO, the Submitter cited four specific concerns: (1) the certified AP1000 shield building does not meet the NRC's seismic margin requirements; (2) there is no adequate demonstration that the shield building meets the GDC-2 requirements; (3) the NRC's conclusion that an aircraft missile would not penetrate the shield building is not logical and; (4) the shield building wall is insufficiently strong and ductile to resist earthquakes or aircraft missiles.

In the DPO, the Submitter reiterated the concerns expressed previously in the non-concurrence and placed an increased emphasis on concerns with the aircraft impact resistance of the shield building.

2.4 References

2(a) ACRS, "REPORT ON THE FINAL SAFETY EVALUATION REPORT ASSOCIATED WITH THE AMENDMENT TO THE AP1000 DESIGN CONTROL DOCUMENT, December 13, 2010, ML103410351

2(b) Letter to Robert Sisk (WEC) from David Matthews (NRC), October 15, 2009 (ML092320205)

2(c) Summary Evaluation Report, "Enhanced Shield Building Structure WEC Electric Company Report APP-1200-S3R-003. Revision 2 AP1000 Design Control Document, Revision 17, October 29, 2010. (ML103080105)

2(d) Ma, J.S., "Dissenting View on the AP1000 Shield Building SER with Respect to the Acceptance of Brittle Structural Module to be Used for the Cylindrical Shield Building Wall, September 27, 2010

2(e) STAFF RESPONSE TO DISSENTING VIEW ON THE SAFETY EVALUATION REPORT FOR THE DESIGN OF THE AP1000 SHIELD BUILDING (SRP SECTION 3.8.4), November 8, 2010

2(f) DPO-2012-002, "The Certified AP100 Shield Building is Unsafe," NRC Form 680, July 6, 2012

2(g) "Report on the Safety Aspects of the Aircraft Impact Assessment for the WEC Electric Company AP1000 Design Certification Amendment Application," ACRS Letter Report, January 19, 2011

3.0 Statement of Concerns

The statement of concerns (SOC) for DPO-2012-002, as agreed to with the Submitter on November 29, 2012, is presented in Appendix A. The agreed-upon SOC includes two overarching issues: (1) Structural integrity concerns associated with design-basis loadings (seismic, strength and ductility) and: (2) Structural integrity concerns associated with aircraft impact. In addition to the agreed-upon specific concerns, the SOC also includes the Submitter's supporting points in the Attachment to the SOC. In accordance with the DPO process, the SOC defined the scope of the DPO Panel's review. The evaluation that follows was organized in accordance with the specific SOC concerns.

4.0 Evaluation

The DPO panel was tasked to:

Perform a detailed review of the technical issues (as described in the SOC – Appendix A) and conduct any document reviews, interviews and discussions necessary for a complete, objective, independent and impartial review

In support of its independent evaluation of DPO-2012-002, the Panel met with the DPO Submitter several times to obtain his perspectives on his concerns and to discuss aspects of the review. The Panel also reviewed the documents, records, and references cited throughout this report and listed in Appendix B, "Records and Documents reviewed by the DPO Panel." The Panel also interviewed the individuals listed in Appendix C, "Stakeholders Interviewed by the DPO Panel," to obtain their perspectives and/or additional information pertinent to the evaluation. The Panel developed a list of email communications from the Submitter along with a description of any actions taken by the Panel in response to the emails. This evaluation is provided in Appendix D, "Summary of Important Email Communications." Appendix E provides a "Crosswalk" between the DPO Issues and the Email Communications.

Finally, the Panel members met amongst themselves numerous times to plan their work, to review and evaluate the results and to document their conclusions and recommendations.

Due to the nature and complexity of the Submitter's concerns, the Panel retained two experts to assist us in our evaluation:

Mr. Loring Wyllie is a structural engineer and senior principal with Degenkolb Engineers in San Francisco, CA. Mr. Wyllie is an expert in the American Concrete Institute (ACI) Code and has been in continuous practice as a structural engineer and consultant for 49 years. He has been a continuous member of ACI Committee 318, Standard Building Code, since 1972 (over 40 years), an honorary member of ACI and also an honorary or distinguished member of the American Society of Civil engineers (ASCE). For the past 5 years Mr. Wyllie has been leading a team from Degenkolb Engineers which is part of the design team for the Uranium Processing Facility at the Department of Energy's Y-12 Site in Oak Ridge, Tennessee. Mr. Wyllie has used the ACI 349 code for the design of the new Uranium Processing Facility. He assisted the Panel with aspects of the Submitter's concerns regarding design basis loadings and the ACI Code.

Dr. Joe Rashid is the founder and president of ANATECH Corporation. Dr. Rashid has over 45 years of experience in the engineering analysis of complex structures, specializing in three-dimensional finite element modeling and rate effects on material and structural behavior. Dr. Rashid is a Fellow of the American Society of Mechanical Engineers (ASME) and a member of the American Nuclear Society (ANS). He holds a Professional Nuclear Engineer License in the State of California. Dr. Rashid is a leading authority in the evaluation of aircraft impact analyses (AIAs) for new nuclear power plants. Dr. Rashid assisted the Panel in the evaluation of the Submitter's concerns related to aircraft impact.

Both Mr. Wyllie and Dr. Rashid produced reports [4(a,b) and 4(c)] related to the tasking in their specific technical areas. Their reports were key inputs to the Panel's evaluation. However, their reports do not necessarily represent the opinions of the Panel.

References:

4(a) "Expert Report of Selected Questions, AP1000 Enhanced Shield Building," Loring A. Wyllie, Jr., June, 2013

4(b) "My Report on AP1000 Design Issues," Letter from Loring A. Wyllie, Jr. to Panel member, [redacted] September 4, 2013

4(c) "Expert Opinion on AP1000 Shield Building Aircraft Impact Assessment," Dr. Joe Rashid, June, 2013

4.1 Safety Concern #1 – The certified design of the AP1000 shield building does not meet the NRC's seismic margin requirements.

4.1.1 Introduction

The Submitter stated that the shield building wall does not meet the NRC seismic margin requirements due to insufficient thickness of concrete in the wall to resist in-plane shear generated by an earthquake. The Submitter's concern is that the in-plane shear demand exceeds the ultimate in-plane shear strength (capacity) and therefore, high confidence of low probability of failure (HCLPF) is significantly lower than the review level earthquake (RLE) of 0.5g (Item 7 in SOC Attachment, Appendix A). In support of this argument, the Submitter provided a demand and capacity curve for the in-plane shear in the shield wall due to the design basis Safe Shutdown Earthquake (SSE) loading [4.1(a), Page 16]. On the basis of this curve, the Submitter believes that the certified AP1000 shield building does not meet the NRC Seismic Margin Assessment requirements because the in-plane shear stress due to the RLE exceeded the ultimate in-plane shear strength for the entire height of the wall.

a,c

4.1.2 Evaluation Approach

The documents reviewed by the Panel included: (a) NRC's Policy on Seismic Margin Requirements [4.1(b)], (b) the WEC evaluation of seismic margin described in Section 19 of the AP1000 DCD [4.1(c)], (c) the staff's safety review of seismic margins described in the AP1000 Final Safety Evaluation Report (FSER) [4.1(d)], and (d) the WEC design of the shield building described in the design report and its appendices [4.1(e)]. A description of the Submitter's views and how they differ from the staff position is provided. The Panel performed a simple calculation of HCLPFs for the shield building wall using the seismic demands and capacities that were reported in the shield building design report. As a result of its review of the seismic margin issue, the Panel identified additional issues beyond the DPO review that are discussed in Section 4.1.3.

4.1.2.1 NRC's Policy on Severe Accident and Seismic Margin Requirements

The NRC has issued requirements and guidance for addressing severe accidents in several documents. 10 CFR Part 52 codifies some parts of the guidance in the Severe Accident Policy Statement and the Standardization Policy Statement. Specifically, 10 CFR 52.47 (27) requires an application for design certification to contain a description of the design specific probabilistic risk assessment (PRA) and its results. The staff provided recommendations to the Commission for addressing PRA and severe accidents for new reactors in SECY-93-087. The Commission approved the following staff recommendations relating to seismic margin requirements, in Section II.N of the Staff Requirements Memorandum (SRM) to SECY-93-087, "Policy, Technical, and Licensing Issues Pertaining to Evolutionary and Advanced Light-Water Reactor (ALWR) Designs" dated July 21, 1993. The seismic margins methodology is designed to demonstrate sufficient margin over SSE to ensure plant safety and to find any "weak links" that might limit the plant shutdown capability to safely withstand a seismic event larger than the SSE.

> *"PRA insights will be used to support a margins-type assessment of seismic events. A PRA-based seismic margins analysis will consider sequence-level High Confidence, Low Probability of Failures (HCLPFs) and fragilities for all sequences leading to core damage or containment failures up to approximately one and two thirds the ground motion acceleration of the Design Basis SSE."*

New reactor designs are required to satisfy the Commission's expectation (as expressed above in SECY-93-087) that the plant HCLPF value will be at least one and two-thirds times the SSE (i.e., 1.67 times 0.3 g = 0.5g).

4.1.2.2 WEC Evaluation of Seismic Margin HCLPF

In Section 19.55 of the AP1000 DCD, Rev. 19 [4.1(c)], WEC provided a summary of seismic margins with HCLPF for the shield building components in Table 19.55.1 and concluded in-part that:

"The seismic margin assessment (SMA) is used to demonstrate margin over the SSE of 0.3g. Consistent with SECY-93-087, the goal of the SMA is therefore to demonstrate that the plant HCLPF is at least 0.5g peak ground acceleration (pga)."

Also, In Section 11.2 of the Shield Building design report [4.1(e)], WEC provided a summary of seismic margin with HCLPF for the shield building components and stated:

"The seismic margin evaluation is based on a 95 percent confidence of less than a 5 percent chance of failure defined as the HCLPF capacity. A review-level earthquake equal to 0.5g has been established for the seismic margin assessment, and is used to demonstrate margin over the AP1000 Consolidated Seismic Design Response Spectra (CSDRS) earthquake of 0.3g."

In Table 19.55-1 of the DCD, and Section 11 of the Shield Building Report, the following HCLPF values are reported for the shield building structural components:

Components	HCLPF in DCD Table 19.55-1	HCLPF in Section 11.2 of SB Design Report
Tension Ring	0.73 g	a,c
Air Inlet	0.71g	
Conical Roof	0.71g	
PCS Tank	0.81g	
S-C/RC Connection	>0.67g	

The Panel notes that the lowest HCLPF of 0.67 g for the RC Cylindrical Wall is greater than that for the review level earthquake equal to 0.5g. The Panel also noted a discrepancy in HCLPF values reported in the DCD and the shield building design report for the same structural component (e.g., for the conical roof, HCLPF of 0.71g versus 1.11g). In subsequent Panel discussions, with WEC, they acknowledged the discrepancy and committed to enter this item in their corrective action program. The Panel recommends that the staff follow-up on this item.

4.1.2.3 NRC Staff's Final Safety Evaluation Report (FSER)

In section 19 of the FSER, the staff provided its safety review of the seismic margin as follows:

"The AP1000 satisfies the expectation of the Commission as expressed in SECY-93-087 that the plant HCLPF value will be at least one and two-thirds times the SSE. Therefore, the staff concludes that seismic risk for the AP1000 design is acceptable."

4.1.2.4 Panel's Evaluation

WEC determined the maximum in-plane shear force in the shield building for the design basis seismic SSE excitation to be [ ] a,c The shield building design report in Appendix H (page H-63) stated that:

[

] a,c

WEC, in the AP1000 DCD, Rev. 19 (page 3.8-60) also stated that the AP1000:

"Design for in-plane shear is in accordance with the requirements of ACI 349, Chapters 11 and 14. The steel faceplates are treated as reinforcing steel, contributing as provided in Section 11.10 of ACI 349."

The staff provided the basis for accepting the in-plane shear capacity in the FSER (page 3-148):

"The staff finds that although there were concerns regarding the test setup at Purdue, the test results indicate that the design for the in-plane shear strength criteria using [(10 square root of fc')] is adequate. The staff finds that although the Purdue test specimen was actually a framed shear wall and the stiffness of the frames was added to that of the wall during the test, the test results [4.1(f)] help assure the staff of the behavior of the SC wall module under SSE loads"

Note: fc' is the compressive strength of the concrete
h and d are the characteristic dimensions of the cross sectional area

The Panel's expert, Mr. Wyllie concluded that:

"the in-plane shear capacity of the SC wall should have been calculated considering the in-plane shear strength of the [ ] a,c thick steel plates using AISC N690 but was conservatively computed using ACI 349 with a limit of [ ] a,c For in-plane shear, the ACI 349 equations are probably not directly applicable as the [ ] a,c provide for more shear capacity than a normal reinforced concrete wall. Using ACI 349, shear capacity would be limited to 8 $\sqrt{f_c}'$ *hd by Chapter 21. WEC used a capacity of [ ] a,c for shear. The true in-plane shear strength is much greater and dependent on the number of shear studs and tie-rods to locally provide bond of the steel plates to the concrete core"*

Based on the above discussion, the Panel concludes that there is sufficient margin (demand to capacity ratio, i.e. 233/277 = 0.84) in the shield wall design for the in-plane shear demand from the seismic SSE loading of 0.3g.

The Panel also concludes that WEC's determination of the shield wall in-plane shear capacity of [ ] a,c for the design basis SSE load is appropriately conservative. In computing the HCLPF for seismic margin for the review level earthquake, a realistic in-plane shear capacity should be used. WEC estimated a value of [ ] a,c for the in-plane shear capacity on page H-99 of the shield building design report. As stated by WEC, [

] a,c

The Panel notes that for SSE loads, WEC conservatively ignored the contribution of the [ ] a,c and used the in-plane shear design capacity of the shield building wall as [ ] a,c based on the [ ] a,c WEC's estimate of the expected in-plane shear capacity based on Japanese tests that considers only the inner and outer steel plates is [ ] a,c WEC supported the Japanese test based in-plane shear capacity with a computer model prediction (Figure H.4-8 of the shield building design report). The computed in-plane shear capacity of [ ] a,c in Appendix L of the Shield Building Report (page L-93) was based on empirical equations supported by test data. The Panel is of the view that it is prudent to reduce the estimated in-plane shear capacity to account for uncertainties. Due to the potential uncertainties in the test program and the uniqueness of the SC structure, the Panel conservatively reduced the in-plane shear capacity by half [ ] a,c The computed in-plane shear force demand in the shield building wall due to SSE [

] a,c

is lower than the reduced in-plane shear capacity of [ ] a,c The in-plane design basis shear capacity curve shown in Section 4.1.1 (in blue) envelopes the in-plane shear seismic demand curve (in green) for the SSE. The in-plane shear capacity of [ ] a,c not shown in the Figure) will envelope the RLE demand. This consideration also leads to a more realistic SSE design margin [ ] a,c for in-plane shear force.

The Panel computed a HCLPF value based on the realistic in-plane shear margin using the same the methodology documented in Section 11.2.7 of the shield building design report.

The in-plane shear margin factor = [ ] a,c

The HCLPF is calculated as follows:

HCLPF = margin factor x ductility factor x SSE

In the industry practice, a ductility factor of 1.25 is used when a probabilistic fragility analysis is not performed. The Panel conservatively used a ductility factor of 1.0, i.e., no ductility. Considering the ductility factor of 1.0 and the above in-plane shear margin factor, the shield wall HCLPF value for the in-plane shear is computed.

[ ] a,c

4.1.3 Additional Related Issues

As a result of the review of the seismic margin issue, the Panel identified additional issues beyond the DPO review that are not discussed in the SOC.

The Panel noted that the HCLPF values shown in Table 19.55-1 of the DCD were computed prior to the design assessment performed and reported in Appendix L of the shield building design report. The data in this table apparently were not updated to reflect the re-analysis results in Appendix L. In a phone interview, WEC informed the Panel that they do not expect any change in the HCLPF value reported in Table 19.55-1 of the DCD due to the shield building reanalysis reported in Appendix L. WEC did not provide any reference or technical basis for this assumption to the Panel.

The Panel does not have any technical basis to believe that generically, a HCLPF value for a given structural component will remain unchanged when the reanalysis in Appendix L for that component for the design basis combined loads shows a design margin that is lower. The staff reviewed HCLPF calculation methodology is based on the design margin. For example, a HCLPF value of [ ] a,c is reported for the PCS tank in Section 11.3.3.2 of the shield building design report. The HCLPF of [ ] a,c is shown to be calculated using the PCS tank wall design margin of [ ] a,c However, the shield building reanalysis in Appendix L shows the PCS tank wall design margin is reduced from [ ] a,c In this example, the Panel does not have any technical basis to believe that the HCLPF will remain unchanged.

As the Panel noted in Section 4.1.2.2, for some shield building structural components, the HCLPF values reported in Table 19.55-1 of the DCD are at variance with the ones reported in Chapter 11 (pages 16-23) of the shield building design report. The Panel could not find the basis for such variance.

In a phone interview, WEC acknowledged a discrepancy in the HCLPF values for some shield building structural components reported in the DCD and in Chapter 11 of the shield building design report. WEC committed to enter this item in their corrective action program.

4.1.4 Conclusions

Based on the simplified and conservative assumptions for the HCLPF estimate for the shield building wall in-plane shear force, the Panel concludes that the HCLPF exceeds the review level earthquake (RLE) of 0.5g and therefore, the thickness of concrete in the shield wall is sufficient to resist in-plane shear generated by the review level earthquake. The Panel further concludes that the shield building wall satisfies the NRC seismic margin requirements

The Panel also concludes that: (1) WEC did not re-evaluate the HCLPF values consistent with the shield building reanalysis results that are reported in Appendix L and (2) the HCLPF values reported in Table 19.55-1 of the DCD are at variance with the ones reported in Chapter 11 (pages 16-23) of the shield building design report.

The Panel recommends that the staff follow-up on these issues.

4.1.5 References

4.1(a) DPO-2012-002, “The Certified AP100 Shield Building is Unsafe,” NRC Form 680, July 6, 2012

4.1(b) Staff Requirements Memorandum (SRM) to SECY 93-087, “Policy, Technical, and Licensing Issues Pertaining to Evolutionary and Advanced Light-Water Reactor (ALWR) Designs” dated July 21, 1993.

4.1(c) WEC AP1000 Design Control Document, Revision 19

4.1(d) NRC Staff Final Safety Evaluation Report (FSER) ML112091879

4.1(e) WEC Electric Company, APP-1200-S3R-003, Revision 4, Design Report for the AP1000 Enhanced Shield Building, June, 2011

4.1(f) Ozaki, M., S.Akitab, H.Osuga, T. Nakayamad, and N. Adachi, “Study on Steel Plate Reinforced Concrete Panels Subjected to Cyclic In-Plane Shear,” Nuclear Engineering and Design, 228, 1-3, pp. 225-244, 2004.

4.2 Safety Concern #2 – The NRC staff's conclusion that the aircraft missile would not penetrate the AP1000 shield building wall is not logical.

4.2.1 Introduction

Four major technical issues, (a) – (d), were identified by the Submitter under Safety Concern #2.

(a) Out-of-plane shear tests showed that the shield building wall behaved in a non-ductile manner and therefore has insufficient punching shear strength and ductility to resist the aircraft missile. (Item 8 in the attachment to the SOC)

(b) The NRC's conclusion that the aircraft missile would not penetrate through (perforate) the shield building contradicts the aircraft missile impact results for the Economic Simplified Boiling Water Reactor (ESBWR) and the South Texas Project Nuclear Operating Company (STPNOC) amended Advanced Boiling Water Reactor (ABWR) designs that were submitted for design certification. (Item 8 in the attachment to the SOC)

(c) The connection between the shield building roof and the water storage tank is not adequate. Failure of the connection may lead to an impact of the water storage tank onto the containment and the potential compromise of the primary system. (Items 10 and 12 in the attachment to the SOC)

(d) The structural wall may behave in a non-ductile manner under impact loading. (Item 9 in the attachment to the SOC)

Technical issues (a), (b) and (d) are discussed and evaluated in Section 4.2.3, and technical issue (c) is discussed and evaluated in Section 4.2.4.

4.2.2 Background

4.2.2.1 Aircraft Impact Assessment Rule

The aircraft impact assessment (AIA) rule [4.2(a)] is contained in 10 CFR 50.150. Assessment requirements are given in 10 CFR 50.150(a), which states:

"(a) Assessment requirements. (1) Assessment. Each applicant listed in paragraph (a) (3) shall perform a design-specific assessment of the effects on the facility of the impact of a large, commercial aircraft. Using realistic analyses, the applicant shall identify and incorporate into the design those design features and functional capabilities to show that, with reduced use of operator actions:

(i) The reactor core remains cooled, or the containment remains intact, and
(ii) Spent fuel cooling and spent fuel pool integrity is maintained"

WEC performed an aircraft impact assessment and concluded that the shield building would not be perforated due to an aircraft impact and therefore identified the shield building as the key design feature that was relied upon to meet the AIA rule acceptance criteria stated above. The Staff's FSER, Section 19F.1.2 states, *"The applicant credits the shield building as a structure that will remain intact following an impact by a large commercial aircraft. Therefore, containment will also be intact."*

[The Panel notes that the requirements of (i) above, may potentially be satisfied even if the shield building is perforated provided a rigorous assessment of the reactor core or the steel containment is performed to show that their integrity is unaffected by the physical, shock, and fire effects of the impact subsequent to shield building perforation. However, such assessments were not performed since WEC concluded that the shield building is not perforated]

4.2.2.2 Statements of Consideration for the AIA Rule

The Statements of Consideration for the AIA rule (AIA SOC) provide the following clarifications:

(i) "By a "design-specific" assessment, the NRC means that the impact assessment must address the specific design of the facility which is either the subject of a construction permit, operating license, standard design certification, standard design approval, combined License, or manufacturing license application."

The Panel notes that the Tier 1 information that includes the key site parameters for a specific design (i.e., AP1000) constitutes the 'specific design' for the aircraft impact assessment. For AP1000, the key parameters are provided in Table 5.0-1 of the DCD (Rev. 19).

(ii) In performing the assessment, the aircraft rule specifies that ``realistic analyses'' be used. The AIA SOC clarifies that "it (the analyses) must be reasonable and technically acceptable. This can be shown by demonstrating that the analytical techniques being used are generally accepted by the relevant professional/technical practitioners for performing best-estimate analysis for the given application". The AIA SOC further states that "realistic'' is a relative term and is simply intended to avoid requiring the designer to utilize conservative or bounding assumptions in recognition of the NRC's determination that the impact of a large commercial aircraft is a beyond-design-basis event.

(iii) The AIA SOC directs the NRC to focus its review on whether the designer's analyses are within the bounds of known data, known physical phenomena, and use professionally-accepted approaches.

4.2.2.3 NRC and Industry Guidance for AIA

The staff participated with the industry and the Nuclear Energy Institute (NEI) in developing a methodology (NEI 07-13 Rev. 8) [4.2(b)], to assess the effects of an aircraft impact. The aircraft impact assessment methodology is based on analytical techniques and approaches that are reasonable, technically acceptable, and are practiced in the nuclear industry for performing best estimate analysis. The staff, in regulatory guide RG 1.217, "Guidance for the Assessment of Beyond-Design-Basis Aircraft Impacts" [4.2(c)], referenced the NRC-industry developed methodology.

In February 2010, the ACRS also reviewed the draft final RG 1.217 and recommended a revision to the RG to address how uncertainties in the analysis should be considered in demonstrating compliance with 10 CFR 50.150, and to consider the potential for amplification of shock waves in stiff water-filled tanks such as the spent fuel pool [4.2(d)]. In May 2010, the staff addressed the ACRS recommendations in the Final RG 1.217 and Revision 8 of the NEI 07-13.

In summary, the NRC considers an aircraft impact assessment that is based on the methodology in NEI 07-13, Rev. 8, (April 2011) to have met the AIA rule requirements of realistic analysis based on professionally accepted approaches for performing best estimate analysis.

4.2.2.4 WEC Report on the AIA for AP1000

As required by 10 CFR 50.150, WEC performed an assessment of the effects of the impact of a large commercial aircraft on the AP1000 in Appendix 19F to the DCD (Rev. 19).

WEC further stated that the specific assumptions regarding the aircraft impact were based on guidance provided by the NRC and NEI, including the loading function derived from the aircraft impact characteristics for use in assessments of aircraft impact effects.

In particular, in section 19F.3 of the DCD, WEC states that, *"In accordance with the recommendation set forth in subsection 2.4.1(4) of NEI 07-13, Revision 7, an analytical evaluation and experimental verification has been performed for the first of a kind steel-concrete modular design feature subjected to the aircraft impact loading."* Subsection 2.4.1(4) of NEI 07-13 reads as follows:

"Past experience with aircraft impact analysis of nuclear power plant structures has not been all inclusive, and new plant designs may contain design features for which experimental and analytical experience is lacking. In such cases, it is important to recognize that these new design features may be subject to failure modes that are outside the existing experience base, and may require experimentally-verified analytical evaluations. For example, good flexural load carrying capability of a composite steel plate encased concrete wall requires adequate capability to transfer shear across the steel-concrete interface."

In DCD Section 19F.4.1, WEC concludes the following:

"The shield building, as described in Chapter 3, is a key design feature for the protection of the safety systems located inside containment from the impact of a large commercial aircraft. The assessment concludes that a strike upon the shield building would not result in perforation of the shield building so that damage to the containment vessel would not occur. Therefore, the systems and equipment within the containment vessel are not damaged from the impact or from exposure to jet fuel."

4.2.2.5 Staff's Acceptance of the WEC Assessment in the FSER

As a part of the certification and licensing process, in accordance with the AIA rule, the applicant is required to maintain the documentation of the detailed technical assessment and is not required to submit the assessment to the NRC. The applicant is only required to describe the assessment in the DCD at a high level. The staff conducts an inspection of the applicant's aircraft impact assessment at the applicant's site to determine its technical adequacy. The Panel interviewed the staff and their consultant that performed the inspection.

In FSER Section 19F.2.1 (pages 19-52), the staff accepted the applicant's analytical evaluation and experimental verification performed for the SC modular design feature subjected to the aircraft impact loading in accordance with the recommendation set forth in Section 2.4.1(4) of NEI 07-13.

The staff concludes in the FSER that, *"Based on the applicant's use of NRC-endorsed guidance document, NEI 07-13, Revision 7, by qualified personnel, the staff finds that the applicant has performed a reasonably formulated assessment."*

4.2.2.6 PCS Tank and Steel Containment Assessment Performed by WEC

In an interview with WEC, the Panel learned that WEC initially did not consider an aircraft impact on the Passive Cooling System (PCS) tank. The Panel's understanding was that the judgment of the NRC staff was to agree with WEC for not considering an aircraft impact on the PCS. WEC subsequently performed the aircraft impact assessment of the PCS tank, after the ACRS raised questions regarding the assessment. WEC informed the Panel that they conducted an analysis which [ ] a,c Based on their analysis, WEC determined that the PCS tank would not be penetrated. They also confirmed to the Panel that the NRC staff did not review the PCS tank aircraft impact assessment.

Although their analysis results indicated that the PCS tank would not be penetrated, WEC stated that they conservatively performed an assessment of debris generation as a result of aircraft impact and the impact of the debris onto the steel containment in the event that the PCS tank were to have been damaged or breached. This assessment first assumed the generation of approximately [ ] a,c of debris that was then assumed to fall approximately [ ] a,c onto a radiation shield plate supported from the conical roof. The assessment showed that the supporting structure for the radiation shield plate was able to contain the debris and that the support structure did not fail.

WEC also informed the Panel that they performed an additional conservative assessment which assumed failure of the radiation shield plate structure. In this case, the shield plate structure was assumed to [ ] a,c with the shield plate structure [ ] a,c falling approximately [ ] a,c and impacting the top of the containment structure. The assessment showed that the [ ] a,c but not penetrated by the debris. WEC informed the Panel that the NRC staff had not reviewed the steel containment assessment for the potential falling debris.

Although the NRC staff did not review the above assessment, the ACRS pursued the matter further and concluded that the assessment provided reasonable assurance of compliance with the requirements of the aircraft impact rule [4.2(o)]. The ACRS also recommended that the staff should evaluate information and analyses presented to the ACRS to determine if there is a need for further inspections. WEC informed the Panel that staff did not perform further inspections of the analyses presented to the ACRS. The Panel's evaluation for AIA of the design is provided in the following Section (4.2.3).

4.2.3 Panel's Evaluation of Technical Issues (a), (b) and (d)

The Submitter stated in the DPO that the NRC staff's conclusion that the aircraft missile would not penetrate through (perforate) the AP1000 shield building wall is not logical. The Submitter's conclusion is based on the fact that:

"The three feet six inches thick ordinary concrete wall of the reactor building in ESBWR and approximate four feet thick ordinary concrete wall in the reactor building near the spent fuel pool area in STP were penetrated through by aircraft missiles through non-linear computer codes specifically developed for the aircraft missile impact analysis. It is logical to conclude that the three feet thick AP1000 shield building wall would be penetrated through by aircraft missiles because test has shown that the SC concrete wall has less strength, ductility, and energy dissipation/absorption capability than that of a companion ordinary reinforced concrete wall..." (SOC, Attachment Item 8)

In the Submitter's Attachment to the SOC, several references are made to the fact that the SC concrete Panel (element) behaves in a brittle manner and is not suitable to resist aircraft impact. The Submitter states, *"The out-of-plane shear test for the element, with tie bars between the faceplates [ ] a,c failed in a brittle manner with [ ] a,c ACI Code prohibits the use of brittle concrete elements."* The Submitter further states*, "A brittle wall may shatter into pieces when it is struck by impact loading. The shattering effect due to aircraft missile impact on the wall has not been investigated by WEC and the staff."* In a presentation to the Panel and Dr. Joe Rashid on April 23, 2013, the Submitter stated that *the "AP1000 shield building is too thin and too brittle, and possesses too little energy absorption capacity against aircraft impact..."* To support these arguments, the Submitter refers to a comparison of the test results from one of the SC wall Panels with test results for a RC beam-column and the finite element analysis results of Mullapudi, et.al. [4.2(e)] (The Mullapudi results are discussed in Section 4.2.3.2. The Panel will first discuss the comparison of test results and the aircraft impact assessment issues.

4.2.3.1 Comparison of Test Results

The figure below, which was contained in the original DPO submission and also discussed in the Submitter's presentation to the Panel and Dr. Rashid, shows a comparison of results from one of the SC wall panel tests (Purdue tests; red curve) with the test results for a RC beam-column (blue curve) [4.2(f) and (p)]. The Panel reviewed the source information for both curves.

a,c

In a test comparing the strength and deformation behavior of two beams, the Panel would expect that, to make a valid comparison, the type of loads applied to each beam would be the same. However, this was not the case. In the RC beam-column test (blue curve) a compressive axial stress of 590 psi was applied to the cross section of the RC beam which greatly increased the shear strength of the RC beam specimen. [

] a,c significant yielding and ductility would have been observed. Under these conditions the red curve (SC panel test) would have been expected to look similar to the blue curve (RC beam test). The fact that the SOC, the SOC attachment, and DPO filing did not mention that the loading conditions were different in the two tests makes the comparison misleading and the conclusions drawn from the comparison not valid.

The stated objective of the SC panel test program conducted at Purdue was to demonstrate that the out-of-plane shear and flexural capacities of the SC Panel could be calculated using the shear and flexure strength equations in ACI-349. This was successfully demonstrated by the three out-of-plane tests that were conducted at different shear span to depth ratios.

In addition, the SC panel test showed how similar the SC panel's behavior is to that of RC beams. For example, it is well known from hundreds of RC beam tests conducted at different shear span to depth ratios, that when a concentrated load is moved closer to the point of support, the mode of failure changes from a ductile flexural failure to a non-ductile shear failure. This same behavior was clearly demonstrated in the three out-of-plane tests of the SC panel. For a shear span to depth ratio of [ ] a,c a very ductile flexural failure occurred [ ] a,c and for a shear span to depth ratio of [ ] a,c a non-ductile shear failure occurred. The red curve shown in the figure is for a SC panel with a shear span to depth ratio of [ ] a,c and marks a transition between flexural failure and shear failure. In addition, the ductile flexural failure of the SC panel with a shear-span-to-depth ratio of [ ] a,c showed that the SC panel had adequate shear transfer capability across the steel-concrete interface.

There is another aspect of the Purdue SC panel tests that has not been discussed thus far and deserves mention here. The SC panel in the AP1000 forms a curved cylindrical shell. It is not a straight beam or flat plate as are the walls of the ESBWR and STP ABWR reactor buildings. It is the curvature of the AP1000 shell wall, which was not incorporated in the Purdue tests, that significantly reduces the shear stresses in the SC wall under external loading, enhances ductile behavior and allows large amounts of energy to be absorbed. Therefore, in the context of aircraft impact, the test results for a flat SC panel cannot be directly compared to a curved SC panel which is part of a continuous shell. The aircraft impact assessment results (discussed in the next section) clearly demonstrate the superiority of the curved SC panel used in the AP1000 cylindrical shell over the ordinary RC flat construction used for the ESBWR and STP ABWR reactor buildings.

The Panel finds the comparison between the [ ] a,c test and the RC beam test to be misleading because it was never pointed out that the loading conditions were different in the two tests. This skewed the results by making the SC panel appear to be less ductile compared to the RC beam, when in fact, had the loading condition for the SC panel been the same as that for the RC beam, both tests would have very likely produced similar results. Therefore, conclusions drawn from the comparison cannot be considered valid.

4.2.3.2 Aircraft Impact Analysis - Influence of Arching Action

In the Attachment to the SOC (Item 8) the Submitter states: *"The three feet six inches thick ordinary concrete wall of the reactor building in ESBWR and approximate four feet thick ordinary concrete wall in the reactor building near the spent fuel pool area in STP were penetrated through by aircraft missiles through non-linear computer codes specifically developed for the aircraft missile impact analysis. It is logical to conclude that the three feet thick AP1000 shield building wall would be penetrated through by aircraft missiles..."* To support the argument the Submitter, in an email dated February 11, 2013, refers to results of an analytical simulation provided in a paper by Mullapudi, et.al. titled "Impact Analysis of Steel Plated Concrete Walls" [4.2(e)]. In the email the Submitter states that *"Case 4 in the paper is a simulation of aircraft impact on a SC wall with 48 inch thick concrete, 0.75 inch thick faceplates and 0.75 inch diameter tie-bars spaced [ ] a,c apart, and the wall is perforated by the aircraft missile."*

As stated previously, WEC performed a design-specific analytical simulation of the effects on the AP1000 shield building of a beyond design basis impact of a large commercial aircraft and determined that the shield building SC wall was not perforated.

In addition, RES performed an AIA of the AP1000 SB cylindrical shell constructed of SC modules. The RES LS-DYNA model and aircraft impact loading were developed independently of WEC [4.2(q)]. The analysis results showed that the aircraft did not perforate the SB shell. DPO Panel member [redacted] interviewed the RES team who performed the analysis and interactively interrogated the model, the aircraft load definition and results. Based on the review, Panel member [redacted] concluded that the finite element model was constructed in sufficient detail and contained the structural and material characteristics and nonlinear features necessary to produce results of sufficient accuracy.

Based on the WEC and RES results, the NRC staff concluded that the aircraft would not perforate the AP1000 shield building shell, constructed of SC modules, at room temperature. However, the fact that the thicker ESBWR, STP ABWR and the SC wall analyzed by Mullapudi were perforated apparently led the Submitter to conclude that the NRC staff's conclusion that the aircraft missile would not perforate the AP1000 shield building wall was not logical.

The fundamental difference between the three thicker walls that were all perforated and the thinner AP1000 shield building SC wall that was not perforated is the fact that all the walls that were perforated are flat plates, whereas the shield building SC wall that was not perforated is a curved cylindrical shell.

Due to their curvature, thin shells offer important advantages over flat plates. Shell structures permit substantial savings of material (i.e., reduced thickness) compared with designs using flat concrete walls. This *results, "from their ability to translate applied loads into 'membrane' thrusts and shears acting in a plane tangent to the surface at any point. By this means bending and twisting moments, and shears transverse to the surface, are reduced or eliminated" [4.2(g)].* This type of structural behavior, which is unique to shells, is often referred to as "arching action."

It is the arching action of the cylindrical shield building shell wall, more than any other single feature, that explains the fact that the 3 foot thick shell of the AP1000 shield building wall can resist aircraft impact loads without being perforated, while the 3'-6" thick RC flat wall of the ESBWR reactor building, the 4'-0" thick RC flat wall of the STP ABWR reactor building, and the 4'-0" thick SC wall analyzed by Mullapudi, are all perforated by the aircraft impact loading.

The ESBWR and ABWR reactor building walls that were perforated had been analyzed by Dr. Rashid. The Panel contracted with Dr. Rashid to perform a beyond-design basis aircraft impact analysis of the AP1000 shield building SC shell wall similar to the analyses he had performed for the ESBWR and STP ABWR reactor buildings [4.2(h)]. At the April 23, 2013 meeting with the Panel and Dr. Rashid, the Submitter presented his perspectives on the issue to Dr. Rashid.

To demonstrate the significance of shell curvature on response to aircraft impact, Dr. Rashid conducted an analysis *"for two composite Panels, one 60-foot by 190-foot flat Panel, and the same Panel but curved to form a semi-circle. Both Panels have the same design features as the AP1000 SC SB wall, i.e. 6000 psi concrete, 36" overall thickness, two 0.75" steel plates, studs and tie-bars of same size and spacing as the SC SB wall. The analysis utilizes the force-time history loading that has equivalent momentum to that of the airplane used in the analysis of the AP1000 structure... The flat Panel dimensions were chosen to be equal to a case analysed by Mullapudi et.al. paper, (Case No. 4...)..." [4.2(h)]*

In his report [4.2(h)], Dr. Rashid concludes: *"The analysis results... for the flat Panel... show... perforation begins at 0.176 seconds when pieces of the Panel start to break up and fly away." "The analysis of the curved Panel... progressed sufficiently to show that the curved Panel will not perforate." "The purpose of the analysis is not to check Mullapudi's results, but rather to quantify the effects of a curved-wall design relative to a flat-wall design of similar design features. Clearly, the results of Mullapudi analysis cannot be used to represent the design capacity of the AP1000, because it totally ignores the most important features of the design..."* - the arching action effect.

4.2.3.3 Aircraft Impact Analysis of the SC Portion of the Shield Building Wall

The NRC aircraft impact assessment rule defines the impact of a large, commercial aircraft as a beyond design basis event that requires a design-specific assessment of the event for new reactor power plant construction. The EPRI-NEI report NEI 07-13, "Methodology for Performing Aircraft Impact Assessments for New Plant Designs", provides guidelines for performing such assessments and was used as guidance for the AP1000 vulnerability assessment described in Dr. Rashid's report.

Dr. Rashid performed a detailed aircraft impact analysis of the SC portion of the SB structure using an aircraft structural model of equivalent damage potential to the NRC-specified force time history. The analysis utilized ANATECH's explicit dynamics TeraGrande-ANACAPU software system, which was the same software used by Dr. Rashid to perform the aircraft impact assessment for the ESBWR and ABWR reactor buildings.

The finite element model considers the portion of the SB with the largest exposed surface, which has a meridional span of 172 feet, and a circumferential span of 157 degrees; 23 degrees short of a full semi-circle. Because the present analysis deals with the vulnerability of the SB to the punching-shear failure mode, which is a local mode that is largely independent of the global force deflection characteristics of the impacted structure, a 180 degree span is assumed. Symmetric boundary conditions are assumed at 0 and 180 degrees, the base is fixed, and at the top boundary two different conditions are assumed - fixed and free. The finite element model of the SC SB wall has 254,496 nodes and a total of 322,720 elements consisting of 136,800 concrete solid elements, 91,200 steel liner solid elements, and 94,720 stud and tie bar beam elements. The finite element plane model has 7,394 nodes and 17,559 shell and point mass elements.

4.2.3.4 Dr. Rashid's AIA Results and Conclusion

The analysis results for the two cases analyzed, which bound the damage potential from above and below for the punching shear mode, show no evidence of perforation. Based on Dr. Rashid's own experience supported by analysis, he concludes that, *"the AP1000 SC shield building is competent to stop a large commercial aircraft without perforation of the wall or threatening the integrity of the containment."* The Panel concurs with this conclusion.

4.2.3.5 Additional Related Issues

There are some issues that are either not discussed in the SOC and attachment to the SOC or only discussed in the SOC attachment. The issues are: (1) the effect of exterior temperature, (2) experimental verification of the WEC model, and (3) constitutive laws. These issues are addressed by the Panel below.

4.2.3.5.1 Aircraft Impact Analysis - Effect of Exterior Temperature

This issue was not part of the SOC or the attachment to the SOC, but was brought up by the Submitter in a meeting with Dr. Rashid and the Panel on April 23, 2013. However, the Panel considered it important to address the issue. WEC performed the aircraft impact assessment at room temperature (as did Dr. Rashid and RES) and did not consider a lower bound exterior temperature in the aircraft impact assessment. The Panel members had differing views regarding consideration of this issue. The views of Panel members [redacted] and Hackett are contained in Section 4.2.3.5.1(a), and Sections 4.2.3.5.1(a)i and 4.2.3.5.1(a)ii. The views of Panel member [redacted] are provided in Section 4.2.3.5.1(b).

4.2.3.5.1(a) Perspectives of Panel Members [redacted] and Hackett

The minimum exterior design temperature of the shield building is -40 degrees F. In a presentation to Dr. Rashid and the Panel on April 23, 2013, the Submitter stated that, *"The AP1000 shield building is designed for minus 40 degrees Fahrenheit, and its ability to resist aircraft impact under cold weather has not been substantiated and is doubtful."* The minimum temperature issue was not raised by the Submitter in the Statement of Concerns or in its attachments. However, the Panel concluded that the issue needed to be addressed.

The AIA rule clearly states that the assessment should be made *"Using realistic analyses..."* The Statements of Consideration for the AIA rule further state that *"...realistic"* is a relative term and is simply intended to avoid requiring the designer to utilize conservative or bounding assumptions in recognition of the NRC's determination that the impact of a large commercial aircraft is a beyond-design-basis event." The NRC considers an aircraft impact assessment that is based on the methodology in NEI 07-13, Rev. 8, to have met the AIA rule requirements of realistic analysis based on professionally accepted approaches for performing best estimate analysis. Section 2.3 of NEI 07-13 Rev. 8 states that, *"The selection of realistic dynamic strength properties and strain failure criteria for steel and concrete is appropriate as the analyses are for beyond-design-basis events and are intended to represent best estimates of material behavior."*

Minus 40 degrees F is a lower bound temperature and results in lower bound material properties. The use of such lower bound properties is not in conformance with the AIA rule, the Statements of Consideration of the AIA rule and NEI 07-13 Rev. 8.

The A572 steel used for the construction of the SC modules exhibits the classic temperature and strain rate dependence for toughness and ductility, characteristic of ferritic steel materials. That is, over a certain range, lower temperatures and higher strain rates can cause a significant decrease in the toughness and ductility of the material. This is illustrated in the following Figure [4.2(i)]:

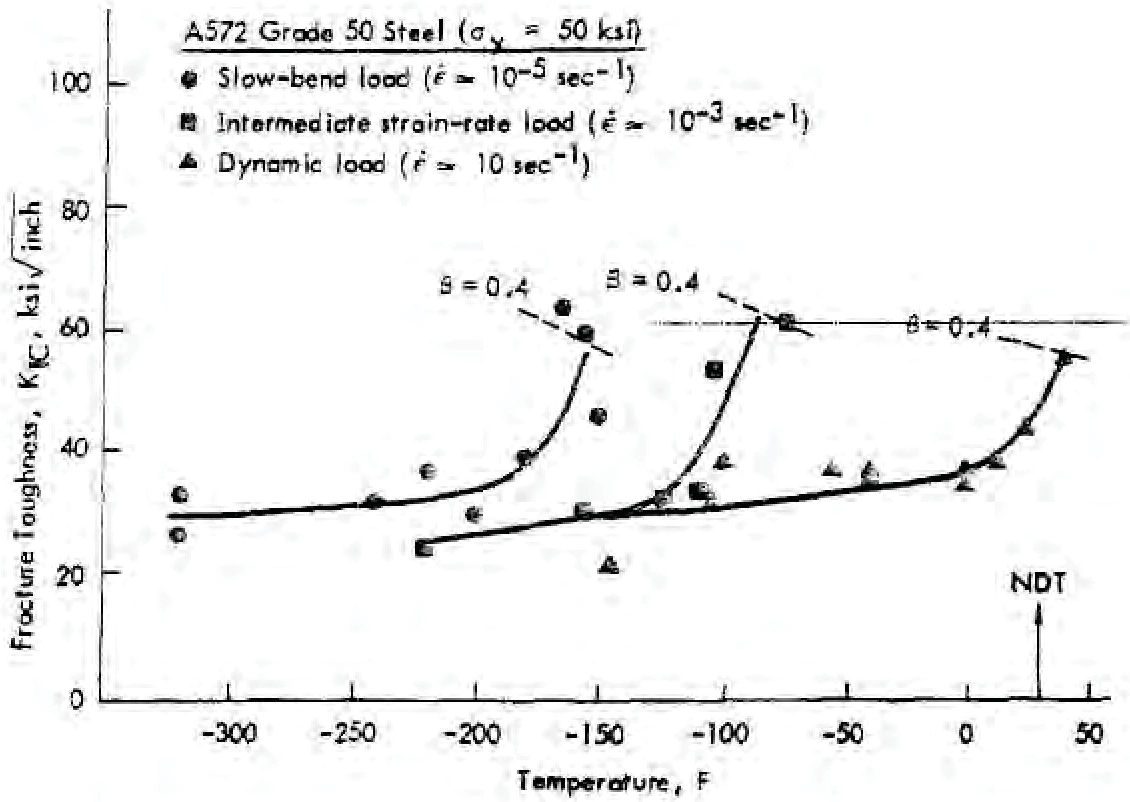


Fig. 9. Effect of temperature and loading rate on fracture toughness of A572 Grade 50 steel (σ_{ys} = 50 ksi)

For dynamic loading, this reference estimates the nil-ductility transition (NDT) temperature for A572 subjected to dynamic loading, to be approximately 30 degrees F. The material exhibits greater toughness above the NDT temperature and it is appropriate and prudent to consider this aspect in design.

As mentioned previously, for a beyond design basis assessment (such as AIA), it is appropriate to use a best-estimate analysis. Ideally, this would be performed probabilistically, considering all of the significant input variables (speed, mass, impact location(s), temperature, etc.). This type of analysis is beyond the scope of the Panel's charter. However, if we restrict our consideration to only temperature, we consider the following assessment to be reasonable:

We assume that an AP1000 could be built and operated anywhere in the United States. Using state-by-state average year-round temperatures for the United States [4.2(j)] yields an overall average of approximately 52 F (a best estimate temperature). As this temperature is significantly above the estimated NDT temperature for A572 Grade 50 steel subjected to dynamic loading, and the material exhibits significant toughness either at or above this temperature, the Panel considers that the WEC analysis, conducted at room temperature, is appropriate and consistent with the AIA rule and the NEI guidance.

4.2.3.5.1 (a)(i) AASHTO Requirements for A572 Grade 50 Steel in Cold Regions

A common application of A572 Grade 50 steel is in highway bridges, which are subject to dynamic impact loads on a daily basis. Many of these bridges are in service in cold regions of the U.S. where temperature extremes can be well below zero.

Based on considerable research and study, the American Association of State Highway and Transportation Officials (AASHTO) adopted in 1973 Charpy V-Notch (CVN) impact-toughness requirements for primary tension members for steel bridges. The toughness requirements for A572 Grade 50 steel is a CVN energy absorption of 15 ft-lbs at a temperature of 70 degrees F higher than the specified minimum operating temperature. [4.2(k)]. For the AP1000 shield building the minimum operating (design) temperature is -40 degrees F. Therefore, to be acceptable for use in highway bridges where the minimum operating temperature is -40 degrees F, A572 Grade 50 steel must absorb at least 15 ft-lbs of energy in a CVN test conducted at 30 degrees F (-40 + 70 = 30).

The figure below shows the CVN energy absorbed at various temperatures for intermediate strain rates and high strain rates associated with dynamic impact. [4.2(k)] For dynamic impact conditions, the figure shows that at the NDT temperature of 30 degrees F, the CVN energy absorbed is approximately 18 ft-lbs. Therefore, A572 grade 50 steel is acceptable for use in the design of highway bridges subject to high strain rate impact loads in regions of the U.S. where the minimum design temperature is -40 degrees F.

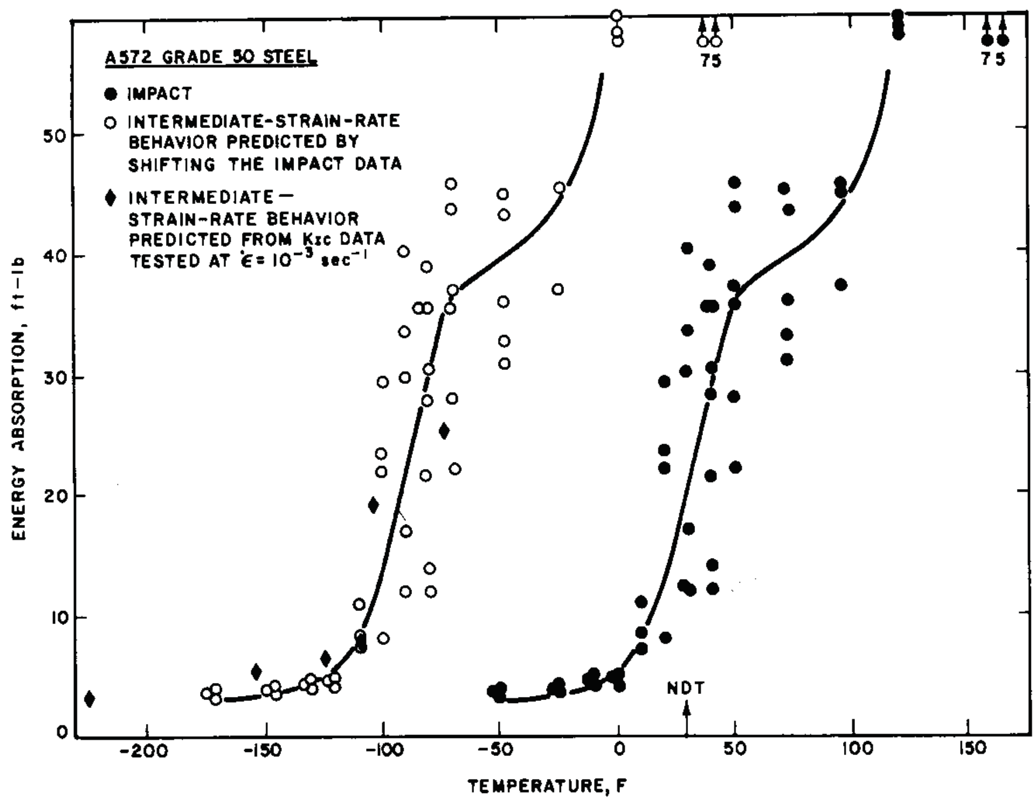


FIG. 15.8. Charpy V-notch energy-absorption behavior for impact loading and intermediate strain-rate loading of standard CVN specimens.

4.2.3.5.1 (a)(ii) Effect of State of Stress

There is no doubt that the effect of exterior temperatures that are below the true NDT temperature will reduce the ductility of the SC panel steel plates. However, it should be recognized that there are other effects, which have not been explicitly considered in the AIAs, that will increase the ductility of the steel plates during aircraft impact. One such effect is related to the stress state in the steel plate caused by the aircraft impact.

The curves in the figure shown above represent only one state of stress. Due to the test conditions that produced these curves, the fracture toughness plotted in the figure represents the uniaxial fracture toughness of A572 steel and is closely related to its uniaxial failure (fracture) strain. It is well known that the failure strain of steel is highly dependent on the state of stress. Tensile states of stress reduce the failure strain while compressive states of stress increase the failure strain. This effect is accounted for in stress analysis using a triaxiality factor.

During aircraft impact, the exterior steel plate will be subjected to through-thickness compressive stresses and the entire SC panel will be subjected to circumferential compressive membrane stresses caused by arching action. These compressive stresses will create a complex triaxial state of stress consisting of both tension and compression which will increase the failure strain and the associated fracture toughness of the steel plates. In turn, this will result in a lower NDT temperature.

Given that the curves in the figure do not represent the relationship between fracture toughness and temperature for states of stress that include compression, using these curves will lead to conservative results with respect to the possible fracture of the exterior steel plate during aircraft impact. The Panel recommends that if the effect on ductility of temperatures below the NDT temperature is to be considered, then other effects, such as state of stress, that also influence ductility, also be considered.

4.2.3.5.1(b) Perspectives of Panel Member [redacted]

In the following, an alternative perspective of Panel member [redacted] is provided on the issue identified in Section 4.2.3.5.1 regarding the consideration of cold outside temperature in conjunction with the generic AIA. The Submitter stated that the ability of the shield wall to resist aircraft impact under cold weather has not been substantiated and is doubtful. The additional perspective provided below is intended to communicate information to aid in a well-informed DPO decision.

The Panel determined that WEC performed the generic aircraft impact assessment at room temperature and did not consider the effect of outside cold temperature on the shield wall material. It is recognized that the temperature of -40°F, the "design basis" outside temperature of the shield wall, is not realistic for the generically certified beyond design basis AIA. However, the aircraft impact assessment for the certified design at room temperature is also not realistic since it is a well-recognized physical phenomenon that the fracture toughness of the shield wall steel plate is decreases due to cold temperature. The fracture toughness is the inherent property of the material that determines its ability to absorb energy from the aircraft impact load. The WEC heat transfer analysis of the shield building shows that [

] a,c

The behavior of the material whether it is ductile or brittle at a certain temperature is indicated by the nil ductility temperature (NDT). The NDT is the temperature above which a material is ductile and below which it is brittle. Once a material is cooled below the NDT, it displays a loss of ductility and is less ductile on impact compared to higher temperatures. This phenomenon leads to a reduction in material failure strain. The NDT for the A572, Gr. 50 steel for a high loading rate similar to the aircraft impact type of dynamic load is at or about freezing, i.e., 32°F (shown in Figure in Section 4.2.3.5.1). The AIA analysis of a certified standard design is required to be a realistic and best estimate analysis. A realistic analysis requires a consideration of contributing physical phenomena. The best estimate analysis requires the best estimates of input parameters, e.g., outside cold temperature to best estimate the impact toughness behavior of the steel plate to the aircraft impact load. The best estimate (mean) of outside cold temperatures in about one-third of the United States in winter months (December, January, and February) is below NDT, the freezing temperature of 32°F.

The NRC's review of the AIA is required to be focused on whether the designer's analyses are within the bounds of known data and known physical phenomena. I believe the use of room temperature is not within the bounds of known cold temperature data and it ignores the known physical phenomena that the fracture toughness of certain metals at cold temperature is reduced.

The shield building for the AP1000 employs modular steel and concrete sections. This type of construction has not previously been used in the US for nuclear industry. The industry experience with complex non-linear dynamic analysis for aircraft impact on such first-of-a-kind nuclear power plant structures is lacking. In my view, the generic WEC AIA analysis for the certified standard design based on at room temperature (70°F) is not a realistic and best estimate analysis because it does not consider the best estimate of one of the key physical parameter -the outside cold temperature, that is below the freezing temperatures in about one-third of the United States during the winter months. It is noteworthy that the aircraft impact rule does not require the applicant of a combined operating license (COL) to perform an aircraft impact assessment if it references a certified standard design that has generically determined that the AIA rule requirements are met.

On the basis that: (1) the generic analysis of the aircraft impact assessment in the DCD is based on room temperature rather than at best estimated cold temperature, (2) the ability of the shield wall to absorb impact energy (fracture toughness) is reduced at below freezing temperatures, and (3) the aircraft impact rule does not require a COL applicant of a certified design to perform an AIA, I agree with Submitter's concern, "the ability of the shield wall to resist aircraft impact under cold weather has not been substantiated and is doubtful." From my perspective, the vulnerability of the aircraft impact on AP1000 design in about one-third of the United States during winter months is indeterminate and could potentially challenge the Panel's conclusion that the aircraft missile would not perforate the SB wall. I recommend that the staff should follow-up on the issue.

4.2.3.5.2 WEC Analytical Evaluation and Experimental Verification

The staff acceptance of the WEC AIA is based on its finding that the applicant has performed a reasonably formulated AIA (FSER Section 19F.3). The staff provided its basis for concluding that the AIA is reasonably formulated in FSER Section 19F.2.1. The staff relied on WEC confirmation that the, *"AIA is based on the guidance of NEI 07-13, Revision 7 and that the AP1000 assessment does not take exceptions to portions of NEI 07-13 guidance that apply to the AP1000 design."* The applicant further stated that an analytical evaluation and experimental verification has been performed for the first-of-a-kind steel-concrete modular design feature subjected to the aircraft impact loading in accordance with the recommendation set forth in Section 2.4.1(4) of NEI 07-13. Based on the applicant's use of the NRC-endorsed guidance document, NEI 07-13, Revision 7, the staff concluded that the applicant has performed a reasonably formulated assessment.

The Panel performed an independent and objective evaluation of whether WEC has met the intent of NEI 07-13 subsection 2.4.1(4) with regards to analytical evaluation and experimental verification (benchmarking of computer models) for the first-of-a-kind steel-concrete modular design feature subjected to the aircraft impact loading, Subsection 2.4.1(4) of NEI 07-13 states:

"Past experience with aircraft impact analysis of nuclear power plant structures has not been all inclusive, and new plant designs may contain design features for which experimental and analytical experience is lacking. In such cases, it is important to recognize that these new design features may be subject to failure modes that are outside the existing experience base, and may require experimentally-verified analytical evaluations. For example, good flexural load carrying capability of a composite steel plate encased concrete wall requires adequate capability to transfer shear across the steel-concrete interface."

4.2.3.5.2.1 Benchmarking Background

The ACRS describes the benchmarking of the LS-DYNA code to experimental data performed by WEC In a letter dated January 19, 2011 [4.2(n)]. The letter states:

"WEC provided comparisons of the predictions of the LS-DYNA model with an experiment on a beam representing a SC structure with greater tie bar spacing under high out-of-plane shear loads." [ ] [a,c] *"The load-deformation behavior predicted by the model agreed well with the results of the experiment; the comparison adequately supports the use of the model for these analyses."*

"The AIA analysis included comparisons of the predictions of the LS-DYNA model with penetration tests conducted in Japan on SC structures. The predictions show adequate agreement with the tests. Although the geometry of the specimens in these tests differs from that of the shield building, the comparisons support the use of the model to predict local failures associated with aircraft impact."

The Panel notes that although the assessment was reviewed by the ACRS, the Committee did not conduct a quantitative verification of the WEC analyses.

Subsequent to the WEC AIA of the shield building (SB), two independent AIAs of the SB were performed. One AIA was performed by RES using a refined LS-DYNA model. The RES results showed that the aircraft did not perforate the SB. The Panel notes that RES did not perform a static or dynamic LS-DYNA benchmark of steel plate and concrete composite panels like the SC module. However, RES has performed over a dozen LS-DYNA benchmark analyses of hard and soft missile impacts into reinforced concrete and pre-stressed concrete slabs, as well as, pre-test benchmark prediction analyses. Based on these LS-DYNA benchmarks performed by RES, the Panel has confidence in RES' ability to adequately model the SC wall of the SB and perform aircraft impact analyses. The other AIA was performed by the Panel's expert Dr. Rashid, who had performed the AIAs for the ESBWR and STP ABWR. Using a different proprietary finite element program specifically developed for aircraft impact, Dr. Rashid also concluded that the SB was not perforated (See Section 4.2.3.4). The two AIAs performed independently by others provides some assurance as to the ability of the WEC model to produce reasonable results

4.2.3.5.2.2 Panel's Independent Evaluation

The Panel interviewed the staff and WEC in order to get clarity and a better understanding of the benchmarking of computer models that WEC used for the AIA. Based on interviews with WEC, the Panel understands that [

] [a,c] It was not clear to the Panel how WEC translated these adjustments to the AIA LSDYNA model.

It is the Panel's understanding that [

] [a,c] In the Panel's opinion, benchmarking to global displacement results alone does not constitute an adequate benchmark for a complex highly nonlinear dynamic model. The Panel believes benchmarking should also be done against the strains within the components of the tested specimen to meet the intent of NEI 07-13 Subsection 2.4.1(4).

Based on the above, the Panel concludes that it is not able to determine that WEC has met the intent of NEI 07-13 subsection 2.4.1(4) with regards to benchmarking of computer models for AIA.

4.2.3.5.3 Constitutive Laws

In Item 1 of the attachment to the SOC the Submitter comments on the constitutive laws for RC and SC concrete elements. The Submitter states that:

"Constitutive laws for a reinforced concrete element are the mathematical relationship between the internal strength, ductility, and energy dissipation/absorption capability of the element under external applied loads. These constitutive laws are required to form the basis for analysis methods or computer codes for computing the behavior of a structure..." The Submitter also states that "Constitutive laws of ordinary reinforced concrete elements are known after decades of physical testing and analytical research" and that the "constitutive laws have not been established for the SC concrete element due to the lack of testing data." This last comment is restated by the Submitter in Item 12 of the SOC attachment; "it is almost impossible or much less possible to estimate the SC wall's behavior due to the lack of information on constitutive laws for the SC concrete elements."

From the above comments the Panel notes that the Submitter has defined *"constitutive laws"* as *"the mathematical relationship between the internal strength, ductility, and energy dissipation/absorption capability of the element."* This is not correct. Constitutive laws are simply the relationship between stress and strain in a material model.

For the static and seismic analyses of the SB, WEC used linear elastic material models for RC and SC module components. [

] a,c The method of transformed section is appropriate where adequate shear transfer can occur between the concrete and steel. This adequacy is well established for RC members and has been demonstrated for SC panels by the Purdue tests for the [ ] a,c This method was used to transform the RC and SC components to create the NI05 ANSYS model discussed in Section 4.3(a) of the report.

When performing a nonlinear analysis, as in the case of AIA, the constitutive laws for RC and SC modules become more complex. A nonlinear finite element analysis of RC members is typically carried out using elements with separate material models (constitutive laws) for reinforcing steel and concrete, which are then combined with bond-slip elements at the interfaces of the two constituents to create the composite RC element. The techniques for constructing finite element models composed of composite materials, such as RC, are well known and proven, *"after decades of physical testing and analytical research,"* to use the Submitter's words.

Composite materials and composite construction have been used in civil and aerospace structures for decades. The technical literature is rich with articles and texts on the subject (e.g. [4.2(l)]). There is nothing unique about the SC module composite construction that does not allow programs such as LS-DYNA, ABAQUS and ANSYS to accurately model and analyze such structural systems. RC composite beams have been analyzed successfully for decades using these programs. Steel beam/concrete slab composite bridges have been analyzed using finite element models for decades as well. In this type of construction the steel beam is anchored to the concrete slab using Nelson studs; the [ ] a,c to anchor the steel plate to the concrete in the SC module. All that is necessary is for the stud spacing be adequate to allow shear transfer to take place between the steel plate and concrete. [
] a,c

The same proven techniques used to develop RC finite element models are used to develop the finite element model of the SC module. Dr. Rashid has spent over 40 years developing concrete constitutive laws and developing composite finite element models. Again, the techniques used to construct composite models of the SC module are the same as those used to construct the composite models of RC. The constitutive laws of all components: the concrete, the steel, the Nelson studs, the shear ties, have been well known and continuously improved through decades of testing and analytical research, as are the techniques for connecting them to form the composite component.

For the reasons discussed above, the Panel does not agree with the Submitter's statement that *"it is almost impossible... to estimate the SC wall's behavior due to the lack of information on constitutive laws for SC concrete elements."*

4.2.4 Panel’s Evaluation of Technical Issue (c)

The Submitter identified the following technical issue in Safety Concern #2:

> (c) The connection between the shield building roof and the water storage tank is not adequate. Failure of the connection may lead to an impact of the water storage tank onto the containment and the potential compromise of the primary system.- (Items 10 and 12 from the SOC attachment)*

The Panel reviewed the integrity of the PCS tank and the steel containment due to the aircraft impact.

4.2.4.1 PCS Tank Assessment

As previously mentioned in Section 4.2.2.6, WEC performed the aircraft impact assessment of the PCS tank after the ACRS raised questions regarding the assessment. WEC informed the Panel that they conducted an analysis which [ ] [a,c] Based on their analysis, WEC determined that the PCS tank would not be penetrated. They also confirmed to the Panel that the NRC staff did not review the PCS tank aircraft impact assessment.

WEC did not discuss whether other failure modes had been investigated. One such failure mode could involve the tank outer wall being sheared off at the connection to the conical roof, and portions of the tank dropping through the annulus. This is a slightly different failure mode than postulated by the Submitter, but would effectively lead to the same result with portions of the tank dropping through the annulus.

The Panel believes that such a shear failure mode may be possible based on the following reasoning. In a direct horizontal impact on the side of the PCS tank, the impact footprint would envelop a considerable portion of the tank. At the instant of impact the water, being incompressible, would act as a rigid inertial mass. The combined weight of the tank concrete and water is more than an order of magnitude greater than a large commercial aircraft. As a result, during most of the impact duration, the tank would barely move. However, the transfer of momentum by the impulse generated by the impact would give the inertial mass of the tank and water an initial velocity and momentum which would have to be resisted, or decelerated, by the shear forces at the connection between the tank wall and the conical roof. A simple hand calculation using the impulse-momentum equation shows that the impact may generate sufficient force to create an average shear stress at the base of the outer wall that is in excess of the ACI-349 allowable shear stress of [ ] a,c

The Panel concludes that it cannot provide an independent opinion regarding reasonable assurance of the validity of the WEC assessment of the PCS tank in view of: (1) the results of the Panel's simple quantitative in-plane horizontal shear demand to capacity calculation, (2) the Panel's observations with respect to the out-of-plane shear reinforcing details of the tank connection to the roof [Section 7.4.4], (3) the fact that [ ] a,c and (4) the fact that the NRC staff did not inspect and review the WEC PCS tank aircraft impact assessment.

The Panel recommends that the staff should follow-up on this issue.

4.2.4.2 Steel Containment Assessment

In a phone conversation with the Panel, WEC clarified that they had performed an assessment of the steel containment for potential debris falling on it from the PCS tank. WEC also stated that it had performed the steel containment assessment for a drop of the [ ] a,c radiation shield onto the containment from a [ ] a,c WEC confirmed to the Panel that the description of the design of the [ ] a,c radiation shield that is supported above the top of the steel containment, is not provided in any of the documents submitted to the staff for the design certification. WEC also informed the Panel that the NRC staff has not inspected the steel containment assessment for the potential falling debris [4.2(m)].

The Panel cannot provide an independent opinion on the reasonable assurance of the validity of the WEC assessment of the steel containment's integrity due to the impact of PCS tank debris or the impact of the [ ] a,c radiation shield in view of the fact that: (1) the Panel could not perform its own independent assessment of the impact of the radiation shield plate on the steel containment due to a lack of design information for the shield plate and its support system, and (2) the fact that the NRC staff had not inspected and reviewed the WEC assessment of the PCS tank debris falling onto the containment shell and had not inspected and reviewed the WEC assessment for the drop of the [ ] a,c radiation shield plate onto the containment.

Considering the fact that the WEC AIA of critical structures (PCS tank and the steel containment) has not been reviewed and verified by the Panel or the NRC staff, the Panel recommends that the staff perform a review of these assessments.

4.2.5 Conclusions

4.2.5.1 Technical Issues (a), (b) and (d)

The out-of-plane shear tests (Purdue tests) showed that the SC panel has the same failure mode characteristics as RC beams. At low shear-span-to-depth ratios both fail in a non-ductile manner (shear failure); at larger shear-span-to-depth ratios both fail in a ductile manner (flexural failure); and at intermediate shear-span-to-depth ratios there is a transition between ductile and non-ductile failure modes.

In Section 4.2.3.2 the Panel's expert, Dr. Rashid, demonstrated the significant effect that shell curvature has on the ability of the SC panel to resist aircraft impact. It is this curvature, which is the reason why the SC panel of the SB can survive an aircraft impact without perforation while thicker SC flat panel and RC walls are perforated by the aircraft.

Based on the Panel's review of the aircraft impact analysis performed by Dr. Rashid, the Panel concurs with his conclusion that*, "the AP1000 SC shield building is competent to stop a large commercial aircraft without perforation of the wall or threatening the integrity of the containment."*

With regard to the effect of external cold temperature on the integrity of the shield wall due to an aircraft impact:

Panel members [redacted] and Hackett consider that the WEC evaluation of this issue was appropriate and in accordance with the NEI AIA guidance regarding use of best-estimate properties for the beyond design basis assessment (Section 4.2.3.5.1(a)). Panel member [redacted] provided his perspective and technical bases in Section 4.2.3.5.1(b) and he agrees with the Submitter's concern stated as, "the ability of the shield wall to resist aircraft impact under cold weather has not been substantiated and is doubtful." He recommends that the staff should follow-up on this issue.

With regard to benchmarking of computer models for the AIA:

The Panel concludes that it is not able to determine that WEC has met the intent of NEI 07-13 subsection 2.4.1(4) with regards to benchmarking of computer models for AIA.

4.2.5.2 Technical Issue (c)

The Panel concludes that that it cannot provide an independent opinion on the reasonable assurance of the validity of the WEC assessment of the PCS tank and the steel containment assessment for potential debris impacts. The Panel recommends that the staff should follow-up on the WEC aircraft impact assessment of the PCS tank and evaluation of debris impact onto the steel containment.

4.2.6 References

4.2(a) AIA Rule, 10 CFR 50.150 and associated SOC

4.2(b) NEI 07-13, Rev. 8 (ML111440006)

4.2(c) Regulatory Guide 1.217 (ML092900004)

4.2(d) ACRS review of RG 1.217 (ML101170344)

4.2(e) T.R.S. Mullapudi, P. Summers, I.H Moon, "Impact Analysis of Steel Plate Concrete Wall, "ASCE Structures Congress 2012.

4.2(f) J.K. Wright and M.A. Sozen, "Strength Decay of RC Columns under Shear Reversals," ASCE Journal of the Structural Division, May 1975.

4.2(g) "Design of Concrete Structures," George Winter and Arthur Nilson, McGraw-Hill, Ninth Edition, 1979.

4.2(h) “Expert Opinion on AP1000 Shield Building Aircraft Impact Assessment.” Dr. Joe Rashid, June 2013

4.2(i) "Fracture and Fatigue Control of Structures," S.T. Rolfe and J.M. Barsom, Prentice-Hall, Inc., 1977.

4.2(j) National Oceanic and Atmospheric Administration (NOAA) website – United States State by State year round average temperatures.

4.2(k) S.T. Rolfe, ”Fracture and Fatigue Control in Structures”, Engineering Journal, American Institute of Steel Construction, First Quarter/1977.

4.2(l) J.R. Vinson and T.W. Chou, "Composite Materials and Their Use in Structures," Halsted Press, John Wiley and Sons, 1975.

4.2(m) NRC Staff’s AIA Inspection for AP1000 (ML120980583)

4.2(n) “Report on the Safety Aspects of the Aircraft Impact Assessment for the WEC AP1000 Design Certification Amendment Application,” ACRS Letter, January 19, 2011

4.2 (o) “INVESTIGATION ON IMPACT RESISTANCE OF STEEL PLATE REINFORCED CONCRETE BARRIERS AGAINST AIRCRAFT IMPACT PART 1: TEST PROGRAM AND RESULTS”, by Jun Mizuno, Yoshikazu Sawamoto, et., al., 18th International Conference on Structural Mechanics in Reactor Technology (SMiRT 18), Beijing, China, August 7-12, 2005

4.2 (p) "Back-up Calculation for Comparison of RC Specimen 40.033A to SC Specimen with a/d = 3.5 at N72," by Y.L. Mo and Thomas T.C. Hsu, University of Houston.

4.2(q) Email from Jose Pires to [redacted] dated February 6, 2014, Subject: AP1000 AIA Assessment.

4.3 Safety Concern #3 – The certified shield building wall does not possess sufficient strength and ductility to resist an earthquake and/or aircraft missile impact loading

4.3.1 Introduction

The technical issues identified in Safety Concern #3 that, “The certified shield building wall does not possess sufficient strength and ductility to resist an earthquake and/or aircraft missile impact loading that is specified by the NRC, are further elaborated as follows:

(a) Seismic demand was under predicted and the actual demand required for the design will be higher due to WEC not having considered the following: (Item 11 from the SOC attachment)

   (i) The inherent torsion due to the irregular (Zigzag) boundary conditions.

   (ii) The demand~~s~~ from accidental torsion required by the American Concrete Institute (ACI) Code and the Standard Review Plan (SRP), NUREG-0800.

   (iii) The analysis for the ACI Code requires consideration of second order P-Delta effects. The water tank resting on top of the shield building, containing approximately 7 million pounds of water, will add to the P-Delta effect.

(b) The Steel-Concrete (SC) module Capacity is determined incorrectly due to the following:

   (i) The ACI Code design equations are not applicable to the SC module elements used in the AP1000. (Item 1 from the SOC attachment)

   (ii) WEC ignored the ACI Code's procedures, criteria and requirements, which were endorsed by the NRC, for the design and review of concrete buildings. (Item 2 from the SOC attachment)

   (iii) The NRC staff misinterpreted the ACI Code's ductility requirements for concrete elements for seismic design as being "no requirement." That is "The Code does not specify a ductility level nor does it specify that ductility should be in every single structural component of the structure." (Item 3 from the SOC attachment)*

   (iv) The NRC staff accepted the SC module element even though when laboratory tested, it failed in a brittle manner and did not meet the acceptance criteria established jointly by the NRC and WEC.

   (v) The NRC staff accepted a non-linear static push over analysis *in lieu* of the ACI code required testing as a basis for the shield building SC module design. Push over analysis is inappropriate and inadequate. NRC improperly accepted push over analysis neglecting test results. The pushover analysis method is prohibited by building codes for use on an irregular, tall, or safety important building such as the shield building. Therefore, the NRC failed to understand that the nonlinear static pushover analysis is not applicable to the AP1000 shield building. The Nuclear Installation Inspectorate (NII) of the United Kingdom concluded that the pushover analysis is not applicable to the shield building, and both NII and Brookhaven National Laboratory (BNL) discovered many problems with the WEC analysis and results. (Items 4, 5 and 6 from the SOC attachment)

(vi) WEC did not consider the ACI code capacity for in-plane shear strength reduction due to interaction with out-of-plane shear forces.

4.3.2 Evaluation of Technical Issues

4.3.2 (a) i: The design did not consider inherent torsion due to the irregular (Zigzag) boundary conditions

The Submitter's concerns regarding inherent torsion are documented in the DPO filing dated July 5, 2012 [4.3(a)] and the SOC [Appendix A]. Further clarification of the concerns was provided by the Submitter in an Attachment to the SOC and an email to the Panel dated April 11, 2013 [4.3(b)]. Excerpts from these two documents are provided below:

SOC Attachment Item # 11:

The following analyses were not performed in obtaining the demand of forces or stresses in the shield building wall for earthquake loading: (1) forces or stresses generated due to the irregular shape of the restraining (supporting) system between the steel plate portion and the ordinary reinforced concrete portion of the shield building wall both in the circumferential and meridional directions, (2) accidental torsion effect,...

Analysis, testing, and building failure data after earthquakes have indicated that an irregular supporting system would generate torsional forces or stresses in the structure compared with that if the supporting system is symmetrical. For the AP1000 shield building, the portion of the SC shield building that contains SC concrete elements, is irregularly restrained or supported by the ordinary reinforced concrete portion of the shield building both in the circumferential and meridional directions. These irregularities were not well captured in the mathematical model, and their effect on forces or stresses in the shield building wall is not well unaccounted for.

The Submitter's email to the Panel dated April 11, 2013 [4.3(b)] stated:

"...due to the zigzag shape of the boundary (support) conditions that anchor the SC portion of the shield building, the AP1000 shield building becomes torsionally irregular in both vertical and horizontal directions. Therefore, the torsional modes and their associated torsional force (stress) in the entire SC portion of the shield building should be obtained through analysis. However, the in-plane shear stress due to the torsional modes caused by the torsional irregularity as a result of the zigzag shape of boundary (support) conditions was not calculated

by WEC, and therefore needs to be calculated and added to the demand of the total in-plane shear stress of the wall so that the thickness of the wall and the amount of reinforcement can be designed to satisfy that demand."

"The mathematical model, as shown in the attachment,... resembles the actual boundary (support) conditions of the SC portion of the shield building because (1) only the concrete of the SC portion sits on the top surface of the RC portion of the shield building and is anchored to the RC portion of the shield building through steel dowel bars, and (2) the bottom of the exterior faceplates is not anchored to or even toughing the top surface of the RC portion of the shield building, and therefore no force (stress) can be transferred from the faceplates to the RC portion of the shield building." "[T]he use of a rectangular SC element representing the whole three feet wide (thick) wall and a rectangular RC element representing the whole three feet wide (thick) wall and the assumption that both elements are connected together at the boundary (support) location in the WEC mathematical model is improper and incorrect because it eliminated the torsional irregularity for the entire SC portion of the shield building and the resulting torsional modes and their associated torsional force (stress) during earthquakes. WEC did not perform the analysis for the inherent torsion due to the torsional irregularity as a result of the irregular (zigzag) shape of boundary (support) conditions for the SC portion of the shield building."

WEC Description of the Seismic Analysis - Section 3.2.1 of the Shield Building Design Report

"In order to obtain the proper boundary conditions between the Shield Building cylinder and the other portions of the Nuclear Island, the Shield Building cylinder is modeled with the Nuclear Island that includes the Auxiliary Building and containment internal structure. The NI05 three dimensional model shows fine mesh, with each element approximately 5 feet square. [

] a,c

Staff's Acceptance of the WEC Assessment in the FSER

The staff discusses the analysis and design of the shield building in Sections 3.8.4.1.1.3.2 and 3. The staff concludes that, *"the design approach involving the three levels of analysis to determine the load magnitudes (seismic demands), the member forces, and deformation demands and including confirmatory analyses, provides a logical, reasonable, and adequate technical approach to developing the shield building design and, therefore, is acceptable."*

The Panel's expert, Mr. Wyllie [4.3(c)] provided the following assessment regarding inherent torsion due to the zig-zag boundary condition:

"WEC has performed numerous computer analyses of the Shield Building and the attached Auxiliary Building. Figure 3.2-1 (page 3-9) is one example. The design summary of this analysis is given in Section 3.2.2.12 (pages 3-20 to 3-27) and illustrates the effects of the

Auxiliary Building on the Shield Building. WEC's Level 1 analyses is the [ ] [a,c] finite element model shown on page 3-9. They then performed a Level 2 analysis with a refined [ ] [a,c] finite element model plus a confirmatory analysis using a large [ ] [a,c] finite element model discussed in Appendix B. Additional analyses with detailed thermal effects are discussed in Appendix L. All of these results clearly show the effects of the stiffness of the Auxiliary Building on the Shield Building. (Note – Figures and pages refer to the shield building report)

From my review of these analysis results, I believe the inherent torsion due to irregular boundary conditions of the Shield Building has been adequately studied and provided for in the design of the Shield Building."

4.3.2 (a) i: Panel's Independent Evaluation

General Discussion

The cylindrical shell of the shield building (SB) intersects the rectangular walls and floors of the auxiliary building (AB). The intersection of the AB with the SB forms a continuous series of vertical and circumferential lines. These intersection locations where the SB is integrally connected to the AB are also the locations where the SC portion of the SB cylindrical wall transitions to RC and are referred to by the Submitter as the "irregular zigzag or step shape" boundary condition.

Inherent torsion occurs during a seismic event as a result of the inherent physical, geometric and material properties of a building when the center of resistance does not coincide with the plan location of the center of mass. A properly constructed finite element dynamic model that accurately incorporates the physical, geometric and material properties of the auxiliary and shield buildings (ASB) and the connection between the elements of the SB and the elements of the AB at their intersection (the zigzag boundary) can accurately capture the response behavior of the ASB including the effects of inherent torsion and the resulting in-plane shear forces in the SB wall. Inherent torsion is typically not explicitly discussed in analysis calculations and design reports, and consequently, there is no discussion of inherent torsion in the DCD or FSER.

Evaluation

There are three statements in the Submitter's clarification email of April 11, 2013 [4.3(b)] that summarize the essence of the issues related to inherent torsion. These are:

> (1) *"...the in-plane shear stress due to the torsional modes caused by the torsional irregularity as a result of the zigzag shape of boundary (support) conditions was not calculated by WEC..."*
> (2) *"... the bottom of the exterior faceplates is not anchored to or even touching the top surface of the RC portion of the shield building, and therefore no force (stress) can be transferred from the faceplates to the RC portion of the shield building."*
> (3) *"... the use of a rectangular SC element representing the whole three feet wide (thick) wall and a rectangular RC element representing the whole three feet wide (thick) wall*

and the assumption that both elements are connected together at the boundary (support) location in the WEC mathematical model is improper and incorrect because it eliminated the torsional irregularity for the entire SC portion of the shield building and the resulting torsional modes and their associated torsional force (stress) during earthquakes."

Evaluation of Statement (1):

The "torsional irregularity" as a result of the zig-zag shape of the boundary connecting the SC and RC portions of the SB wall is created by the difference in stiffness between the SC and RC portions of the wall and the fact that the zig-zag boundary [ ] a,c Due to the steel faceplates' high stiffness in shear and the higher concrete strength of the SC portion of the SB wall, the SC portion of the SB wall has about a [ ] a,c than the RC portion of the wall. Because of the higher stiffness of the SC portion compared to the RC portion, the [ ] a,c than the [ ] a,c which results in a shift of the center of resistance in the lower elevations of the SB. This shift, in turn, makes the SB unsymmetrical or irregular and creates additional torsional moments during a seismic event. As briefly discussed in Section 3.2.1 of the Shield Building Design Report (page 3-4), and quoted earlier in this section, the increased stiffness of the SC portion over the RC portion was calculated by WEC [

] a,c

Evaluation of Statement (2):

The Design Report Figure 4.1-2 (page 4-6) and Appendix H Figure H.2-1 (page H-54) show the details of the mechanical connection between the SC portion of the SB and the RC portion [ ] a,c The SC/RC connection has been designed to [ ] a,c from the SC portion to the RC portion of the SB wall. The fact that the steel faceplates are, *"not anchored to or even touching the top surface of the RC portion of the SB"* concrete has no bearing on the ability of the connection to transfer forces and moments from the SC portion to the RC portion of the SB wall.

The most critical forces that must be transferred across the SC/RC boundary during a seismic event are meridional tension, meridional compression, and in-plane shear.

With respect to meridional compression, the termination of the faceplates at the cold joint creates a structural discontinuity at this location. However, because concrete is strong in compression, all of the meridional compressive stresses in the faceplates will be transferred to the concrete [ ] a,c Based on St. Venant's principle [4.3(f)],

[

] a,c

[

] a,c

While concrete is strong in compression, it is weak in tension, and the mechanism that transferred the compressive stresses from the faceplate to the concrete cannot be relied upon to transfer meridional tension. To transfer the tensile stresses in the faceplates to the RC portion of the SB wall, WEC provided [

] a,c

The in-plane shear stresses in the faceplates are transferred in the same way as the meridional compressive stresses [

] a,c

Based on the Panel's examination of the SC/RC connection details the Panel does not support the Submitter's concern that at the SC/RC connection, *"no forces can be transferred from the faceplates to the RC portion of the SB."* In addition it is the Panel's expert judgment that the rotational stiffness, axial stiffness and shear stiffness at the cold joint would be approximately the same as it would be if the cold joint had existed between two RC portions of the SB wall.
[

] a,c

Evaluation of Statement (3):

To determine whether the SB shell finite element model has been adequately modeled to accurately capture the internal forces necessary to design the structure, three things must be considered: (1) the element's ability to represent the internal forces, (2) the element's ability to represent the geometry, and (3) the refinement of the element mesh.

The elements used in the NI05 ANSYS finite element model to represent the three foot wide SC and RC portions of the SB wall are fully integrated thin shell elements that produce a linear distribution of normal strain and constant shear strain within the element and incorporate through thickness shear deformation. Thus, the element can capture all the necessary internal forces. The geometry of the three foot thick SC and RC walls can be accurately modeled with thin shell elements because the R/t ratio (shell radius to thickness ratio) is greater than 20, where the standard for the cut-off between thin shell and thick shell analysis is an R/t ratio of 10.

[

] a,c This ratio is sufficiently small for a linear element to accurately capture the meridional bending moment at structural discontinuities. Therefore, the element mesh is sufficiently refined to accurately capture the frequencies and mode shapes, and associated moments, shears and axial forces for design of the SB. [

] a,c

For a linear elastic seismic analysis, it is the Panel's expert judgment that the element type (fully integrated thin Shell), element mesh and connectivity at the SC/RC connection will produce accurate results for design.

Conclusion

It is the Panel's expert judgment that the WEC finite element models have been constructed to adequately replicate the physical, geometric and material properties of the AB and SB structures, including the connectivity of the SB to the AB along the zig-zag boundary and the difference in stiffness between the SC and RC portions of the SB. The Panel, therefore, concludes that the effects of the dynamic interaction between the AB and the SB, and the effects of inherent torsion due to the difference in stiffness between the SC and RC portions are appropriately reflected in the seismic analysis results for the SB wall.

4.3.2 (a) ii: The design did not consider the accidental torsion due to seismic excitation required by the ACI Code and the Standard Review Plan (SRP), NUREG-0800.

The Panel reviewed the SRP guidance, WEC analysis report, and the expert's evaluation related to the accidental torsion issue. The Panel did not find in the staff's FSER, a staff evaluation of the accidental torsion due to seismic excitation.

WEC consideration of accidental torsion - Section 3.2.1 Page 3-4 of the shield building design report

[

] *a,c*

directions of input are combined by the square root of the sum of the squares (SRSS) method and include a factor of 1.05 on the horizontal components to account for accidental torsion.

The equation for this seismic combination SRSS is shown below:

$$[\,(\alpha A_{NS})^2 + (\alpha A_{EW})^2 + (A_{VT})^2\,]^{1/2}$$

where,

A_{NS} *= maximum element forces due to safe shutdown earthquake (SSE) response analysis in X (north-south)*
A_{EW} *= maximum element forces due to SSE response analysis in Y (east-west)*
A_{VT} *= maximum element forces due to SSE response analysis in Z (vertical)*
α *= factor due to torsion effect in North-South or East-West (1.05)*

The Panel's expert, Mr. Wyllie, stated [4.3(c)]:

"Accidental torsion is required by NUREG-0800 Section 3.7.2 which requires an additional eccentricity of ± 5 percent of the maximum building dimension applied both horizontal directions. Accidental torsion is not specifically required in ACI Codes but is required in ASCE-7 or whatever load provisions accompany the ACI Code. WEC has applied the 5 percent accidental torsion as stated in Section 3.2.1 (page 3-4). They increased the load by 1.05 on the horizontal components to account for accidental torsion. The Shield Building with its circular plan is inherently resistant to torsion. I believe accidental torsion has been adequately considered in the design of the Shield Building."

The Panel did not find the Staff's evaluation of the accidental torsion due to seismic excitation in the FSER.

Panel's Independent Evaluation

Accidental torsion is introduced to account for building torsion arising from discrepancies between the mass, stiffness, and strength distributions used in analysis and the true distributions at the time of an earthquake. It also accounts for torsional vibrations introduced by a rotational component of ground motion, and other sources not considered explicitly in the soil-structure interaction analysis.

To account for accidental torsion NUREG-0800, Section 3.7.2 requires that an additional eccentricity of 5% percent of the maximum building dimension be assumed for both horizontal directions separately for each floor elevation. The SRP also states that, *"An acceptable alternative, if properly justified, is the use of static factors to account for torsional accelerations in the seismic design of Category I structures."*

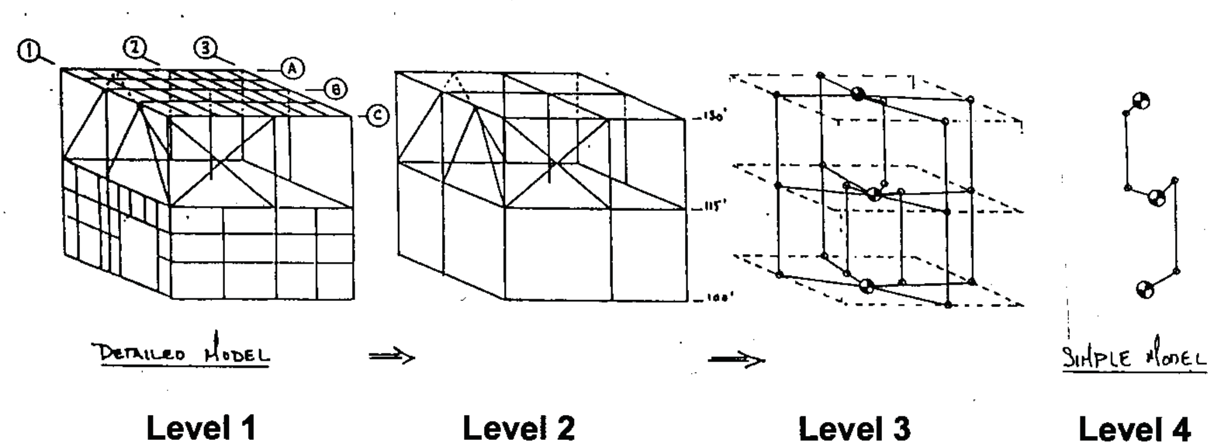
ACI Codes 318 and 349 do not specifically require consideration of accidental torsion.

The [ ] [a,c] NI05 model used by WEC to perform the seismic analysis is a large solid-shell finite element model of the AP1000 nuclear island. In order to obtain the proper boundary conditions between the shield building cylinder and the other portions of the nuclear island, the shield building is modeled with the nuclear island that includes the auxiliary building and containment internal structure (CIS).

All structural components of the nuclear island, including walls, floors, columns and beams, are incorporated in the model. The model captures the stiffness characteristics and distributed mass properties of these components to create an integrated model that can accurately predict the static and dynamic response of the entire structure to various loading events.

In a state-of-the-art finite element model, such as the large WEC [ ] [a,c] NI05 model, the inherent torsion created by the eccentricity of the center of mass from the center of resistance (i.e., the shear center) is already taken into account, as discussed in the previous section. Accidental torsion, on-the-other-hand, is not an inherent property of such a model, but must be accounted for separately.

Implementing accidental torsion in a finite element dynamic model can be challenging and difficult, depending on the level of idealization of the model. The Figure below shows four Levels of idealization of a simple two story building. Level 1 represents a state-of-the-art seismic model like the large WEC NI05 model of the AB, SB and CIS. Level 4 represents the simplest stage of idealization of the same structure. Level 4 (Equivalent beam lumped mass model) has been most commonly used in older seismic models, such as those used to perform the seismic analysis of most of the Category 1 structures of the currently operating nuclear power plants (NPPs).



Levels of Structural Model Refinement from a Detailed Member Specific Finite Element Model to a Single Equivalent Beam Lumped Mass Model.

In a Level 4 seismic model accidental torsion is easily implemented simply by increasing the eccentricity of the center of mass. However, implementing accidental torsion in a Level 1 seismic model is a difficult challenge since the model may contain thousands of solid, shell and beam elements. Implementation requires that the eccentricity of the center of mass be increased at each elevation in a Level 1 seismic model while at the same time not changing the model's stiffness characteristics, location of the center of resistance (CoR) and total mass. To achieve this the density of the elements would have to be changed in such a way that the eccentricity of the location of the center of mass at each elevation increases by 5% of the plan dimensions while keeping the total mass the same. This requires that the elements farthest away from the CoR have their density increased, and elements closest to the CoR have their density decreased without changing the total mass. Complicating matters further is the fact that this process is not unique, and as would be expected, the process is fraught with the possibility for error.

The only reasonable way to avoid introducing such errors into a Level 1 model and satisfy the intent of the accidental torsion provisions of the SRP is to increase the torsional strength of the structure by applying an amplification factor to the seismic response results, which is consistent with the "acceptable alternative" provision of SRP Section 3.7.2. This is exactly what WEC has done [

] a,c

Conclusion

The Panel finds that the WEC approach for implementing accidental torsion is consistent with the "acceptable alternative" provision of SRP Section 3.7.2 and is reasonable and acceptable.

The Panel notes that in the seismic analysis discussed above, the PCS Tank was not evaluated for accidental torsion. WEC explained in a call on July 3, 2013 that the reason the PCS Tank was not evaluated using the seismic results from the global NI05 finite element model was because the PCS tank in the global model did not contain sufficient detail to perform a seismic evaluation of the tank structure. This can be clearly seen from the plot of the NI05 finite element model in Figure 3.2-1 of the Design Report. To evaluate the PCS tank a detailed quarter (90 degrees) model of the SB and PCS tank was constructed and the seismic accelerations from the global response spectra analysis were statically applied to the quarter model. WEC stated that the static analysis of the quarter model produced [ ] a,c internal forces than the response spectra analysis of the global model. Because the internal forces were already [ ] a,c in the SB quarter model, WEC did not include the [ ] a,c in the original SB global response spectra analysis for the purpose of evaluating the PCS tank. The Panel did not verify these analyses. However, based on WEC's description of the analyses they performed, the Panel believes that not including the [ ] a,c is reasonable.

4.3.2(a) iii: The design did not consider the second order P-Delta effects required by the ACI Code

The P-Delta Effect

During a seismic event horizontal seismic accelerations cause a building structure to displace laterally. This results in the location of the force resultants of vertical dead load and vertical seismic inertia loads (the “P”) to be displaced laterally (the “Delta”). The product of the force resultants and the lateral displacements (P x Delta) causes additional bending moments in the structure, which, if the displacements are large enough, can be significant. The creation of these additional bending moments caused in this manner is called the P-Delta effect.

The Submitter’s Concern

The Submitter stated that, the "*ACI code requires a second-order P-Delta analysis to account for the additional forces generated due to the deformed shape of the shield building. Due to the heavy vertical weight of the water tank on top of the roof, the vertical load (P) is considerably larger on the shield building wall compared to that of other structures that do not have a heavy tank on their top, and the forces from the P-Delta effect could by significant. However, WEC did not perform the second order P-Delta analysis, and the forces from the P-Delta effect are not accounted for.*"

Opinion of the Panel’s Expert

The Panel’s expert, Mr. Wyllie [4.3(c)] stated that, *“the ACI 349 Code does not specifically require consideration of P-Delta effects as the ACI code deals more with strength and capacities. The P-Delta effect is addressed in industry standard ASCE/SEI 43-05, “Seismic Design Criteria for Structures, Systems and Components in Nuclear Facilities.” It states in*

Section 3.2.2, Linear Dynamic Analysis, that "P-Delta effects shall be included, if significant." It then defines "if significant" *as "If inclusion of P-Δ effects results in greater than a 10% increase in the imposed moment demand on a structural member, the effects shall be included; otherwise, they may be omitted."*

Based on his review, Mr. Wyllie concluded that*, "predicted deflections for the SSE would not result in an increase of 10 percent in the moments of the Shield Building. For the RLE, he concluded that the predicted deflections could be close to causing a 10 percent increase. However, the Shield Building Design Report indicates that for the RLE, only some local yielding occurs, and thus there is some capacity for stresses from P-Delta if they were not included. He concludes that it was not clear if P-Delta effects were included in the WEC analyses and that at the SSE level of deformation the P-Delta effects, considering the roof top water tank, would be insignificant."*

WEC and Staff Evaluation

P-Delta effects are not discussed in the WEC SB design report and were not discussed in the staff FSER.

Panel's Evaluation

The Panel performed a calculation [4.3(f)] to determine the effect of P-Delta on the forces acting at the base (El. 100') of the SB wall, and to determine if the P-Delta effect would increase seismic response by more than 10%. An idealized model of the SB was developed based on the shape of the first fundamental mode of a shell finite element model of a cylindrical PWR concrete containment structure. The maximum displacement at the top of the idealized model was set equal to the maximum SSE seismic displacement of the SB of [ ] [a,c] from Table 10.2-5 (page 10-39) of the Design Report. The weight of all components making up the SB was calculated, including the SB cylindrical shell, conical dome concrete, liner and steel beams, and the PCS tank and water, and applied vertically at their appropriate locations to the deflected shape of the model at their appropriate locations.

The magnitude of the SB base moment at El. 100' was calculated and the maximum stress resultant determined and compared to the stress resultants from the seismic load combinations in Table 3.2-2 (page 3-7) of the design report. The comparison showed that the increase in the stress resultants at the base of the SB due to the P-Delta effect is less than 1%. Because this is much less than 10%, the effects of P-Delta do not need to be considered in the seismic analysis.

Conclusion

Based on the results of our expert's assessment and an independent calculation, the Panel concludes that the fact WEC did not consider the effect of P-Delta in the seismic analysis to be appropriate and acceptable.

4.3.2(b) i: The ACI Code design equations are not applicable to the SC module elements used in the AP1000, and.

4.3.2(b) ii: WEC ignored the ACI Code's procedures, criteria and requirements, which were endorsed by the NRC, for the design and review of concrete buildings.

Evaluation

With regards to the application of the ACI code to the SC module elements, Mr. Wyllie advised the Panel as follows:

"The design of the Shield Building SC (steel-concrete) wall is based on ACI 349, Code Requirements for Nuclear Safety-Related Concrete Structures. ACI 349 draws most of its requirements directly from ACI 318, Building Code Requirements for Structural Concrete. The structural steel codes, or specifications as they are called, are also applicable to the Shield Building's SC wall. ANSI/AISC N690, Specification for Safety-Related Steel Structures for Nuclear Faculties, is the appropriate steel code.

Neither of these codes is directly applicable to the Shield Building's SC wall. ACI and AISC have an agreement that for composite members, ACI cover's composite columns and AISC cover's composite beams.

The SC Shield Building wall is a composite wall with a concrete core and a steel plate on both faces. Many of the ACI 349 code design equations are applicable as the wall has been designed as a concrete member with the steel plates on each face replacing the reinforcing bars on each face of a normal reinforced concrete wall. Bond between the steel reinforcement (the plates) and the concrete is provided by the studs and tie rods. Design for axial load and flexure, both in –plane and out-of-plane, are directly applicable to ACI 349 design equations using the steel plate area for area of steel. The design equations for out-of-plane shear of ACI 349 are directly applicable with the tie-rods welded to each surface plate acting as shear reinforcement. For in-plane shear, the ACI 349 equations are probably not directly applicable as the [ ] [a,c] provide for more shear capacity than a normal reinforced concrete wall. Using ACI 349, shear capacity would be limited to 8 $\sqrt{f_c'}$ hd by Chapter 21. WEC used a capacity of [ ] [a,c] for shear. The true in-plane shear strength is much greater and dependent on the number of shear studs and tie-rods to locally provide bond of the steel plates to the concrete core. It is my opinion that the [ ] [a,c] used by WEC is conservative for in-plane shear capacity of the SC wall that is used in conjunction with the design basis SSE load."

Conclusion

The Panel finds that WEC did not ignore the pertinent ACI Code procedures, criteria and requirements, and that many of the ACI 349 Code equations are applicable to the SC module elements used in the AP1000.

4.3.2(b) iii: The NRC staff misinterpreted the ACI Code's ductility requirements for concrete elements for seismic design as being "no requirement." That is "The Code does not specify a ductility level nor does it specify that ductility should be in every single structural component of the structure.

Evaluation

In the following sections, Mr. Wyllie [4.3(c),(d)] provided the philosophy and "requirements" of the ACI Code with respect to the ductility, ductility level and failure modes of structural components and whether or not the shield building as-designed will behave in a ductile manner and the technical bases for his opinions.

Ductility Requirements

The NUREG-0800 and ACI 349 require the design of reinforced concrete nuclear safety-related structures to be based on elastic response in the safe shutdown earthquake (SSE). Failure of such structures during shaking more severe than the SSE, or beyond-design-basis earthquake, is substantially reduced by detailing the components for ductile response using provisions drawn from Chapter 21 of ACI 318 and ACI 349.

Earthquake induced lateral loads are primarily resisted by Shield Building walls acting in in-plane shear. Hence, in order for the overall building ductility to be achieved, the Shield Building walls should exhibit a sufficiently ductile in-plane shear response. It is noteworthy that brittle failure of the SC wall element in the Purdue University tests was observed for out-of-plane loads and not for in-plane shear loads. A further discussion of the significance of the ductility in the design is discussed in Section 4.3.2 (b)iii and of the test results is discussed in section 4.2.3.1. Seismic margin methodology is designed to demonstrate sufficient margin over SSE to ensure plant safety. Section 4.1.2.4 of the DPO report shows that the shield building wall has sufficient in-plane shear capacity to accommodate higher in-plane shear force due to a 67% increase in design basis SSE acceleration.

The ACI 349 is the "Code Requirements for Nuclear Safety-Related Concrete Structures". It is based on ACI 318 which is the Building Code for Structural Concrete and supplemented for nuclear facilities. The philosophy and requirements for ductility and failure modes are provided at several levels. First, consider the simple concrete beam resisting gravity loads. For flexure the code provides for proportioning such that the tension reinforcement will yield prior to the crushing of concrete in compression. This allows overloads to cause an increase in the noticeable deflections providing a warning of potential failure. In addition by assigning phi (ϕ)

factors, or capacity reduction factors for shear, a brittle sudden shear failure is normally precluded. This is considered a basic level of ductility by allowing the beam to resist overloads of gravity with warning prior to failure.

ACI 349 does require that "ductility should be in every single structural component of the structure." Paragraph 21.2.1.1 of ACI 349 requires that the reinforcing bar detailing requirements of Chapter 21 shall be the design practice for nuclear plants. These reinforcing bar detailing requirements provide ductility. ACI 349 only adopts the "special" systems portion of Chapter 21 of ACI 318. In other words, ACI 349 excludes the intermediate and ordinary structural systems with reduced ductility that is allowed for non-nuclear structures in regions of the United States with lower seismic hazards.

Ductility when discussing seismic resistance has a much broader definition. For seismic resistance, ductility is the ability of a concrete structure to resist loads beyond its elastic strength such that the structure will undergo inelastic response without collapse. The Design Report for the AP1000 Enhanced Shield Building states that the design of the shield building is governed by seismic loadings. ACI 349 requires that "the reinforcing bar detailing requirements of this chapter (Chapter 21) shall be the design practice for nuclear plants." The Commentary of ACI 349 elaborates this issue as follows:

"Many of the special seismic component provisions of Chapter 21 or ACI 318-05 are adopted for the seismic design of safety-related nuclear structures for three key reasons. First, and as noted previously, the adoption of the special seismic provisions provides substantial assurance that structural integrity will be maintained in the unlikely event of beyond-design-basis earthquake shaking or other unforeseen circumstances. Second, the adoption of the special seismic component provisions provides reinforcing bar detailing requirements consistent with the toughness needs of structural members design for Special Facilities class of structures of the Department of Energy non-reactor nuclear production plants wherein limited inelastic response to design-basis earthquake shaking is permitted. ACI 349 is cited as the design Code by the governing design criteria document of these facilities. Third, adoption of many of the special seismic provisions maximizes the possible compatibility between ACI 318-05 and ACI 349."

Ductility Level

ACI 349 specifies a ductility level consistent with the special seismic resisting systems contained in Chapter 21 of ACI 349. Chapter 21 of ACI 349 only includes the sections for special moment frames and special structural walls and deletes all the other sections of ACI 318 for concrete systems with relaxed reinforcing steel detailing and lower ductility systems. In moment frame concrete structures, the beams, columns and beam-column joints have numerous detailed requirements for the rebar detailing to provide ductile performance. The AP1000 Shield Building does not have these members. It is a shear wall system and, in fact, a shear wall system without openings. Thus, there are neither boundary members nor coupling beams in the AP1000 Shield Building, so the only requirements of ACI 349 *applicable to the shear walls are minimum reinforcing (achieved by the steel plates) and design stress limits.*

The ACI 349 Code requires all members of the lateral force resisting system to be detailed for seismic ductile performance. Other members, such as gravity resisting beams or frames, do not need to be ductile or need special detailing unless the seismic building displacements are sufficient to cause inelastic response in those members.

The ACI 349 requires that seismic detailing requirements of Chapter 21 be the design practice for nuclear plants and that the "special" seismic systems be utilized. ACI 349 and 318 do not give specific ductility numbers within these codes. Rather, they are coordinated within ASCE-7, "Minimum Design Loads for Buildings and Other Structures" which contains a Table 12.2-1, Design Coefficients and Factors for Seismic Force-Resisting Systems with all recognized seismic resisting systems and specifies an "R" value for each system. This is a reduction factor for non-nuclear buildings which allows the maximum creditable earthquake forces to be reduced to design level forces considering the inherent "ductility" of that system based on the detailing requirements (Chapter 21 for concrete systems) of the various material codes. The R value for a special reinforced concrete structural wall is 5. These R values have been established based on considerable engineering judgment and observations of system performance in actual earthquakes and laboratory testing. They may not be directly applicable to a "ductility value" used in some analytical studies.

The ductility level of the special seismic resisting systems of Chapter 21 is indirectly suggested by the R values contained in ASCE-7. The ACI 349 specifies a ductility level consistent with the special seismic resisting systems contained in Chapter 21 of ACI 349. The R values in ASCE -7 are not "ductility numbers" but indirectly suggest ductility levels, or at least relative ductility levels. The ASCE-7 is not intended for nuclear power plants and is not required to follow or implement the ASCE-7 in their seismic design. WEC used appropriate criteria for their design.

Shield Wall Ductile Behavior

"The SC wall as designed will behave in a ductile manner under seismic loading. The wall is detailed equivalent to a special structural wall and has been substantially reinforced for [

] [a,c] Impact loading is a different situation and the impact of a significant object is essentially a beyond design basis event. I believe the SC wall module is actually better equipped to resist an impact loading as it has the steel plate exterior surface to somewhat spread out the load and the tie wires provide some shear reinforcement. If the exterior wall was a conventional reinforced concrete wall, as I gather it was early in the design process, that wall would have no ties or shear reinforcement and would probably be less capable of resisting impact loading. If the impact load is great enough, this could be a non-ductile situation in either case.

For most of the SC wall, the shear reinforcement is provided by minimal tie-rods at about [ ] a,c compared to the [ ] a,c near the roofs of the Auxiliary Buildings. [ ] a,c as illustrated in Figures L.4-23 and L.4-24 of the shield building design report (pages L-52 and L-53). I believe WEC's design is adequate in this respect. If the SC wall was a conventionally reinforced concrete wall it probably would not have closely spaced ties throughout the wall."

Conclusion

The Panel concludes that the ACI 349 Code requires ductility to be in every single structural component of the structure and all members of the lateral force resisting system should be detailed for seismic ductile performance and that the ductility level is consistent with the special seismic resisting systems contained in Chapter 21 of ACI 349. The AP1000 Shield Building satisfies the detailing requirements in Chapter 21of ACI 349 applicable to the shear walls - [ ] a,c and design stress limits, thus ensuring that the shield building wall as-designed will behave in a ductile manner under seismic loading.

4.3.2(b) iv: The NRC staff accepted the SC module element even though when laboratory tested, it failed in a brittle manner and did not meet the acceptance criteria established jointly by the NRC and WEC.

Evaluation

The Panel's expert, Mr. Wyllie [4.3(c)] provided the following opinion regarding the significance of the out-of-plane bending/shear test results for the SC module that failed in a non-ductile manner:

"The out-of-plane bending/shear tests are discussed in Section 7.7 of the Design Report. [ ] a,c The fact that some of these specimens may have failed in a non-ductile manner should not be interpreted that the SC wall is non-ductile. I was involved in some laboratory testing some years ago and I learned one approaches test specimen design completely opposite from structural design. In structural design I can add some additional reinforcement for more capacity to prevent failure. But this approach is not valid for laboratory specimen design as we would never learn anything. One designs a test specimen to fail so important information is gathered. One must also remember that if the SC wall was a conventional reinforced concrete wall there would probably be no ties or shear reinforcement and similar RC test specimens would probably have been less ductile than the SC test specimens."

Mr. Wyllie continues, *"I do not believe these tests suggest the SC wall is non-ductile. [*

] [a,c]

Conclusion

The Panel concludes that the shield building as-designed with SC modules will behave in a ductile manner under postulated design basis loading conditions. The SC modules that did fail in a brittle manner had low shear-span-to-depth ratios for which a brittle (shear) failure would be expected, much as it would for an RC beam element.

4.3.2(b) v: The NRC staff accepted a non-linear static push over analysis *in lieu* of the ACI code required testing as a basis for the shield building SC module design.

Evaluation

The Panel reviewed the shield building design report Section 10, and the Staff's FSER Section 3.8.4.1.1.3.13 to get an understanding of whether the pushover analysis was used for the design basis of the shield wall. The Panel observed that the pushover analysis was not referenced in the AP1000 DCD shield building seismic analysis and design Sections 3.7 and 3.8. The Panel also considered Mr. Wyllie's expert opinion expressed as follows:

Pushover analyses were developed primarily as part of ASCE/SEI 41, "Seismic Rehabilitation of Existing Buildings". It is referred to as Nonlinear Static Procedure and often called "pushover analysis". It is used to identify weak links or members in existing buildings considering both ductile and brittle behavior in the lateral force resisting system. For evaluating existing buildings it is allowed without height restriction and on all types of buildings (e.g., irregular, tall, or safety related important building such as the AP1000 shield building) *… building codes do not allow the pushover analysis method for the design of structures like the AP1000 Shield Building.* [However], *They do not prohibit it for additional or supplemental analysis to study seismic margin for beyond design events.*

A resource paper by the National Earthquake Hazard Reduction Program (NEHRP) recommended Seismic Provisions for New Buildings and Other Structures (FEMA P-750, 2009 Edition, Part 3, Resource Paper 2). This resource paper allow the Nonlinear Static Procedure to be used for regular structures less than 40 feet in height and Occupancy Categories I and II (not nuclear structures) in lieu of regular code design. This Resource Paper is not proposed or ready for code adoption but has been provided for review and study.

The shield building design report describes a form of a nonlinear static pushover analysis that was performed on the AP1000 Shield Building. The Design Report does not state or imply that this pushover analysis replace the ACI 349 design procedure or requirements. WEC designed the Shield Building using ACI 349 and AISC N690 design codes and used a form of a pushover analysis to evaluate the margin within the structure and its ability to resist beyond design basis events. As stated in Section 10.1.4 (page 10-3) of the Design Report:

[

] a,c

The Panel also reviewed the staff's evaluation in the FSER (page 3-160) with regards to the non-linear static push over analysis:

"The applicant performed nonlinear confirmatory analysis to predict the behavior of the shield building up to and beyond design basis seismic loading and assess the potential for collapse. The applicant used its [ ] a,c model of the nuclear island to perform a nonlinear pushover analysis of the shield building. The model included the shield building and the entire auxiliary building. This finite element model did not impose constraints that would force a mode of deformation of the shield building structure. Using this model, the applicant's analysis tracked tensile stresses and strains in the steel faceplates, in-plane and out-of-plane shear deformations and stresses, stresses and strains in the [ ] a,c deformations in the connection regions and stresses and strains in the [ ] a,c in the RC wall below the SC wall. The applicant's analysis explicitly modeled the interaction of the shield building with the roof and walls of the auxiliary building. The applicant's model also did not exclude the possibility of shear failures. Instead, it considered concrete cracking for out-of-plane loads as well as in-plane loads and the subsequent distribution of forces to the steel reinforcement. Since the applicant's verification and validation of the model against its own test data did not capture brittle failures, the applicant tracked the possibility of local onset of such brittle shear failures through the use of limiting strains in the [ ] a,c as well as through the combined use of analysis methods with increasing refinement, that is, the combination of [ ] a,c models.

For its analysis, [ ] a,c In addition, the applicant considered various combinations of the directions and intensity of the seismic loads in the two horizontal directions and in the vertical direction. Under these loading conditions and without constraints in the response modes of the structure the applicant calculated the response of the structure to proportionally increasing loads. Proportional increase of the loads is an *approximation in a static pushover analysis. As the structure yields and the response becomes increasingly inelastic, there is a potential for redistribution of the loads through the height of the structure that may affect the subsequent response mode of the structure. The results of the applicant's analysis show that significant inelastic behavior of the wall, other than concrete cracking, will not occur at the design basis loads and will only start at loads closer to the review level earthquake (RLE). On this basis, loading conditions that deviate significantly from those used by the applicant are not expected up to the SSE and RLE levels.*

The applicant's analysis results showed that the highly stressed regions of the shield building were near structural discontinuities such as the connection to the basemat at the 30.40 m (100-ft) elevation, in the region above the roof of the auxiliary building and at the connection of the SC wall to the RC walls. The analysis predicts yielding initiation through yielding of the faceplates in these regions or yielding of the reinforcing bars in the RC walls below the SC walls. Yielding of the steel faceplates is a result of [

] [a,c]

The results of the pushover analysis confirm that the shield building stresses, strains and deformations remain small at the design basis loads and that significant yielding in the SC wall does not start until loading levels beyond the SSE and of the order of the RLE. The results of the analysis confirm that the high stress areas of the wall with complex states of stress from the combination of high membrane forces and out-of-plane forces are the areas of the wall for which [ ] [a,c] *described in Section 4.3.5.2 of this report, showed that these models exhibit ductile out-of-plane behavior under cyclic loading."*

Based on the Panel's review of Section 10 of the shield building design report, the Staff's FSER Section 3.8.4.1.1.3.13, and its expert's views, the Panel finds that WEC designed the Shield Building in accordance with the ACI 349 and AISC N690 design codes and used a form of a pushover analysis only to evaluate the margin within the structure and its ability to resist beyond design basis events

Conclusion

From the review of the WEC Shield Building design report, expert opinion, and the staff's final safety evaluation report (FSER), the Panel understands that the pushover analysis did not replace the ACI 349 design procedure or requirements; rather it provided an understanding of the beyond design basis response. The pushover analysis only confirmed qualitatively that the shield building stresses, strains and deformations remain small at the design basis loads and that significant yielding in the SC wall does not start until loading levels beyond the SSE and of the order of the RLE.

The Panel therefore concludes that the design basis of the shield building for SSE is based on elastic response and not on a non-linear static push over analysis. In accordance with the staff's

FSER, it did not accept a non-linear static push over analysis *in lieu* of the ACI code required testing as a basis for the shield building SC module design.

4.3.2(b) vi: WEC did not consider the ACI code capacity for in-plane shear strength reduction due to interaction with out-of-plane shear forces.

Evaluation

The Panel agrees with the Submitter's assertion that WEC did not consider the ACI code capacity for in-plane shear strength reduction due to interaction with out-of-plane shear forces. However, the ACI 349 code is silent about in-plane shear strength reduction due to interaction with out-of-plane shear forces. The Panel's expert, Mr. Wyllie, expressed his opinion on this issue as follows:

"the in-plane shear capacity of the SC wall should have been calculated considering the in-plane shear strength of the [ ] [a,c] steel plates using AISC N690 but was conservatively computed using ACI 349 with a limit of [ ] [a,c] The out of plane shear strength is computed correctly. Since the in-plane strength is controlled by the steel plate strength and the out-of-plane strength is controlled by the tie bars and their strength and spacing, the shear strengths in the orthogonal directions is essentially independent and the in-plane strength does not need to be adjusted. True, both mechanisms rely on initial diagonal cracking of the concrete, but these cracks are in different orientations and it is my opinion that adjustments in the shear strength are not necessary because of this situation."

The Panel also notes that in general, the maximums of out-of-plane shear demand and in-plane shear demand do not occur at the same location. For example, the Panel observed from Appendix H (Table H.2-2) to the shield wall design report, that at the location of maximum seismic shear demand (basemat Elevation 100 feet), the shield wall out-of-plane shear demand due to SSE is relatively very small [ ] [a,c] compare to the in-plane shear demand [ ] [a,c] In the Panel's opinion, any reduction in the in-plane shear strength due to negligible interaction with small out-of-plane shear forces is not design significant.

Conclusion

On the basis of the Panel's expert's opinion that (1) in-plane shear strength was conservatively computed, (2) out-of-plane shear strength was correctly computed, (3) the fact that in-plane and out-of plane shear are essentially independent, and (4) the fact that any reduction in the in-plane shear strength due to interaction with out-of-plane shear forces is not required by the ACI 349 code, the Panel believes that such interaction need not be considered and the in-plane shear strength considered in the shield wall design is appropriate.

4.3.3 Conclusions

Overall, the Panel concludes that the AP1000 Shield Building possesses sufficient strength and ductility and is designed to resist a review level earthquake that is larger in magnitude than the SSE (Section 4.1.2.4) and/or aircraft impact loading (Section 4.2).

Conclusions on specific technical issues are summarized below.

Technical Issues:

(a) Seismic demand was under predicted and the actual demand required for the design will be higher due to WEC <u>not</u> having considered the following

(i) The inherent torsion due to the irregular (Zigzag) boundary conditions.

Panel's conclusion: The Panel concludes that the effects of the dynamic interaction between the Auxiliary Building and the Shield Building, and the effects of inherent torsion due to the difference in stiffness between the SC and RC portions are appropriately reflected in the seismic analysis results for the Shield Building wall.

(ii) The demand~~s~~ from accidental torsion required by the American Concrete Institute (ACI) Code and the Standard Review Plan (SRP), NUREG-0800.

Panel's conclusion: WEC implemented demands due to accidental torsion by applying a 5% increase to member forces and moments from the horizontal components of the ground motion. The Panel finds that the WEC approach for implementing demands due to accidental torsion is practical, reasonable and consistent with the "acceptable alternative" provision of SRP Section 3.7.2.

(iii) The analysis for the ACI Code requires consideration of second order P-Delta effects. The water tank resting on top of the shield building, containing approximately 7 million pounds of water, will add to the P-Delta effect.

Panel's conclusion: WEC apparently did not explicitly consider the second order P-Delta effects. However, based on the Panel's independent calculation, the increase in stress resultants at the base of the Shield Building is negligible (less than 1%) due to the P-Delta effect.

(b) The Steel-Concrete (SC) module Capacity is determined incorrectly due to the following:

(i) The ACI Code design equations are not applicable to the SC module elements used in the AP1000. (Item 1 in the SOC attachment)*

Panel's conclusion: The SC Shield Building wall is a composite wall with a concrete core and a steel plate on both faces. Many of the ACI 349 code design equations are applicable as the wall has been designed as a concrete member with the steel plates on each face replacing the reinforcing bars on each face of a normal reinforced concrete wall.

(ii) WEC ignored the ACI Code's procedures, criteria and requirements, which were endorsed by the NRC, for the design and review of concrete buildings. (Item 2 in the SOC attachment)*

Panel's conclusion: The staff accepted the SC concrete element for the shield building wall primarily because many of the ACI 349 code design equations are applicable as the SC wall has been designed as a concrete member with the steel plates on each face replacing the reinforcing bars on each face of a normal reinforced concrete wall.

(iii) The NRC staff misinterpreted the ACI Code's ductility requirements for concrete elements for seismic design as being "no requirement." That is "The Code does not specify a ductility level nor does it specify that ductility should be in every single structural component of the structure." (Item 3 in the SOC attachment)*

Panel's conclusion: The Panel agrees with the submitter in that the ACI 349 requires that ductility should be in every single structural component of the structure. The Panel concludes that the ACI 349 Code requires that all members of the lateral force resisting system should be detailed for seismic ductile performance and that the ductility level is consistent with the special seismic resisting systems contained in Chapter 21 of ACI 349. However, the AP1000 Shield Building satisfies the detailing requirements in Chapter 21 of ACI 349 applicable to the shear walls - [ ][a,c] and design stress limits, thus ensuring that the shield building wall as-designed will behave in a ductile manner under seismic loading.

(iv) The NRC staff accepted the SC module element even though when laboratory tested, it failed in a brittle manner and did not meet the acceptance criteria established jointly by the NRC and WEC.

Panel's Conclusion: the shield building as-designed with SC modules will behave in a ductile manner even though some SC module elements with low shear span-to-depth ratios failed in a brittle manner in laboratory tests. The panel notes that at low shear-span-to-depth ratios, the RC beams also fail in a brittle manner.

(v) The NRC staff accepted a non-linear static push over analysis *in lieu* of the ACI code required testing as a basis for the shield building SC module design. Push over analysis is inappropriate and inadequate. NRC improperly accepted push over analysis neglecting test results. The pushover analysis method is prohibited by building codes for use on an irregular, tall, or safety important building such as the shield building. Therefore, the NRC failed to understand that the nonlinear static pushover analysis is not applicable to the AP1000 shield building. The Nuclear Installation Inspectorate (NII) of the United Kingdom concluded that the pushover analysis is not applicable to the shield building, and both NII and Brookhaven National Laboratory (BNL) discovered many problems with the WEC analysis and results.

Panel's conclusion: The design basis of the shield building for SSE is based on linear elastic response and not on a non-linear static push over analysis. The push over analysis provided a qualitative understanding of the beyond design basis response and did not replace the ACI 349 design procedure or requirements. The use of static push over analysis for purposes other than the design is considered appropriate. The staff's did not accept a non-linear static push over analysis in lieu of the ACI code required testing as a basis for the shield building SC module design.

(vi) WEC did not consider the ACI code capacity for in-plane shear strength reduction due to interaction with out-of-plane shear forces.

Panel's Conclusion: The Panel agrees with the Submitter's assertion that WEC did not consider the reduction in the ACI code capacity for in-plane shear strength due to interaction with out-of-plane shear forces. However, any reduction in the in-plane shear strength due to interaction with out-of-plane shear forces is negligible and is not required by the ACI 349 code, Therefore, the Panel believes that such interaction need not be considered and the in-plane shear strength considered in the shield wall design is appropriate.

4.3.4 References

4.3(a) DPO-2012-002, "The Certified AP100 Shield Building is Unsafe," NRC Form 680, July 6, 2012

4.3(b) Email from J. Ma to DPO Panel dated April 11, 2013

4.3(c) "Expert Report of Selected Questions, AP1000 Enhanced Shield Building," Loring A. Wyllie, Jr., June, 2013

4.3(d) "My report on AP1000 Design Issues," Letter from Loring Wyllie, Jr. to Panel Member [redacted], September 4, 2013

4.3 (e) "Theory of Elasticity," Timoshenko and Goodier, McGraw-Hill, Third Edition 1970, (Page 39)

4.3 (f) "Calculation of P-Delta Effects," Panel member [redacted], February, 2013

4.4 Safety Concern #4 – The certified shield building design does not meet the General Design Criteria (GDC) 2 requirements

4.4.1 Introduction

The DPO Panel's understanding of the Submitter's concern is stated in the SOC as follows.

The concern that the shield building design does not meet the GDC 2 requirements is embodied in safety concerns 1, 2, and 3 from the SOC, and in the overall concern that the AP1000 shield building has the following three major differences with the shield buildings in the operating plants. (Item 12 of the SOC attachment):

(a) The shield building is joined together by two different types of construction (i.e., Reinforced concrete and SC module)
(b) The two different types of construction are joined in a zigzag shape at boundaries in both circumferential and meridional directions.
(c) A water tank with a million gallons of water rests on top of the AP1000 shield building roof.

Subsequent to the Panel's issuance of the final SOC, the Submitter in an e-mail to the DPO Panel dated August 29, 2013 [4.4(a)] communicated his concern regarding the appropriate application of the building codes and design guides. The Submitter focused his concern that the certified AP1000 shield building design does not meet the GDC 2 requirement with respect to the SSE event. In particular, the Submitter stated that the in-plane shears due to the SSE in the shield building wall exceed the recommended in-plane shear design value. The Submitter further stated the following:

"10 CFR, Part 50, Appendix A, GDC 2 – *Design bases for protection against natural phenomena,* states *"Structures, systems, and components important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes,….without loss of capability to perform their safety functions." Whether the design of the AP1000 shield building has met this GDC 2 requirement or not should be judged on whether its design meets the sound structural engineering knowledge and practice, as disseminated in building codes and design guides. Both the building codes and design guides are written by knowledgeable and experienced structural engineers and used by structural engineers for practical design and by regulators for permit or licensing review. The purpose and usage of building codes and design guides, and the difference between the codes and guides, are described below first, and then by using the requirements in building codes and the recommended practice in design guides to demonstrate that the certified AP1000 shield building design does not meet the GDC 2 requirement."*

4.4.2 Evaluation

The Panel's evaluation approach was to review (i) the GDC 2 requirement in the Code of Federal Regulations 10 CFR Part 50, (ii) the guidance documents that NRC staff prepared to reflect the compliance with the GDC 2 requirements, and (iii) the implementation of the staff guidance in the seismic analysis and design of AP1000 shield wall structure. The Panel believes that if the staff guidance for SSE seismic analysis and design of shield building wall are reasonably implemented then the shield building structures should be in compliance with the GDC 2 requirements.

The Panel reviewed requirements of the general design criterion (GDC) 2 in the 10 CFR, Part 50, Appendix A with respect to the SSE event. The GDC 2 requirement– *Design bases for protection against natural phenomena,* states "Structures, systems, and components important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes,….without loss of capability to perform their safety functions." The design bases of these structures, systems, and components shall reflect:

1. The design basis shall reflect appropriate consideration of the most severe earthquakes that have been historically reported for the site and surrounding area with sufficient margin for the limited accuracy, quantity, and period of time in which historical data have been accumulated.

2. Appropriate combination of the effects of normal and accident conditions with the effect of the natural phenomena,

3. The importance of the safety functions to be performed.

4.4.2.1 GDC 2 requirement for Design Basis Earthquakes

The Panel reviewed the staff developed guidance and acceptance criteria (NUREG 0800-SRP 3.7.1, 3.7.2, and 3.7.3) that implements the GDC 2 requirements for earthquakes to assure that they are appropriate and contain sufficient margin such that seismic analyses accurately and/or conservatively represent the behavior of SSCs during postulated seismic events.

SRP Section 3.7.1 describes acceptance criteria for developing the required earthquake loadings consisting of the safe shutdown earthquake (SSE). It describes acceptance criteria for developing seismic design parameters.

SRP Section 3.7.2 describes acceptance criteria for the seismic analysis and modeling of seismic Category I structures and major plant systems.

SRP Section 3.7.3 describes acceptable methods for the seismic analysis of seismic Category I subsystems.

Meeting the acceptance criteria of staff guidance in SRP 3.7.1, SRP 3.7.2, SRP 3.7.3 provides assurance that (1) appropriate methods have been used to determine the required OBE and SSE loadings and the seismic demands of structures, systems, and components (SSCs), which will ensure that they will remain functional within applicable acceptance limits, and (2) structures, systems, and components important to safety (seismic Category I) shall be designed to withstand the required seismic demands and thus, will be able to perform their intended safety function and meet the GDC 2 requirement with respect to the SSE event.

4.4.2.2 GDC 2 requirement for combination of normal, accident conditions and design earthquake (SSE)

The Panel reviewed the staff developed acceptance criteria (NUREG 0800-SRP 3.8.4) that Implements the GDC 2 requirements for the design of all Seismic Category I structures and other Safety-related structures including containment enclosure building (shield building). The staff's acceptance criteria references specific ACI design code and includes appropriate load combinations of the effects of normal and accident conditions with the effect of the natural phenomena (SSE) and corresponding structural acceptance criteria for the load combination.

GDC 2 requires that structures other than containment be designed to withstand the effects of natural phenomena combined with those of normal and accident conditions without loss of capability to perform their safety function. Load combinations and specifications cited in this SRP section 3.8.4 provide an acceptable engineering criterion to accomplish that function.

Meeting these requirements provides added assurance to the staff that safety-related structures will be designed to withstand the effects of natural phenomena and will perform their Intended safety function.

4.4.2.3 GDC 2 requirement for importance of safety functions

The Panel also reviewed the applicable ACI code 349 that is referenced in the staff guidance SRP 3.8.4 for the design of reinforced concrete nuclear safety-related structures. The ACI 349, "Code Requirements for Nuclear Safety-Related Concrete Structures" is based on elastic response in the safe shutdown earthquake. The ACI 349 in its commentary (commentary section R 21.1) differentiates between the performance objectives and design requirements for conventional buildings and the safety-related nuclear structures. The code recognizes that the performance objectives and design requirements are significantly different: significant damage but no collapse in the design earthquake for conventional building structures versus elastic response (no significant damage) in the design earthquake for safety-related nuclear structures.

In recognition of the safety functions, the safety-related nuclear SSC's are analyzed and designed to remain elastic with respect to the SSE event.

The Panel believes that once the staff guidance in SRP 3.7.1, 3.7.2, 3.7.3, and 3.8.4 are met, then the shield building structures is in compliance with the GDC 2 requirements. The Panel reviewed the implementation of the staff guidance in the following.

4.4.2.4 Computation of Seismic Demands

In compliance with the GDC 2 requirements, the computation of seismic demands of structures is expected to be in accordance with the staff guidance in SRP 3.7.1, 3.7.2, 3.7.3. The DPO Panel reviewed section 3.7 and Appendix 3G of AP1000 DCD (Revision 19) that describes the development of the required safe shutdown earthquake (SSE) loading and other seismic design parameters that have been used in the seismic analysis. The Panel also reviewed the criteria and methodologies for the seismic analysis and modeling of seismic Category I structures and major plant systems.

In support of his concern, the Submitter identified an ASCE standard ASCE/SEI 7-10, “Minimum Design Loads for Buildings and Other structures’ with regards to building height limits for buildings that use special structural walls as the sole earthquake-resistant system. The Submitter also identified a design guide published by the National Institute of Standards and Technology (NIST) for the seismic design of special structural walls. The standard and design guides are not intended for application to safety-related nuclear structures. The standard provides minimum load requirements and load combinations for strength design and allowable stress design for conventional buildings. As stated in the previous section, the performance objectives for conventional buildings and the safety-related nuclear structures are significantly different: significant damage but no collapse in the design earthquake for conventional building structures versus elastic response (no significant damage) in the design earthquake for safety-related nuclear structures. The Panel finds that WEC evaluated the shield building wall for the combination of shear, flexure, torsion, and axial (tension or compression) forces during earthquakes (Appendix L, page L-99).

The Panel also finds that the following design characteristics and attributes that the Submitter identified in his concerns (in the SOC) are represented and considered in the shield building structure mathematical model that is used for computing the SSE seismic demands.

(a) The shield building is joined together by two different types of construction (i.e., Reinforced concrete and SC module)
(b) The two different types of construction are joined in a zigzag shape at boundaries in both circumferential and meridional directions.
(c) A water tank with a million gallons of water rests on top of the AP1000 shield building roof.

The Panel also reviewed the staff’s evaluation of the seismic design parameters and seismic analyses of shield building structures in Sections 3.7.1 and 3.7.2 the FSER.

Based on its review of the description of seismic analyses provided in the DCD and the staff’s evaluation thereof, and its own expert judgment, the Panel is of the opinion that the three attributes that the Submitter is concerned with have been included in the shield building seismic model and that SSE demands of structures, including the in-plane shear force in the shield wall, are reasonably computed and therefore, the concerns outlined above in (a)-(c) have been considered in computing the SSE demands.

4.4.2.5 Computation of Design Capacity

The Panel's view of the In-plane shear capacity of the shield wall is provided in Section 4.1 of this report.

Using ACI 349, shear capacity would be limited to $8\sqrt{f_c'}$ hd by Chapter 21 that specifies provisions for seismic design of the reinforced concrete nuclear safety-related structures. WEC used a [ ] a,c The true in-plane shear strength is much greater and dependent on [ ] a,c provide bond of the steel plates to the concrete core. Mr. Wyllie opinioned that the [ ] a,c is conservative for in-plane shear capacity of the SC wall because of the contribution of the [ ] a,c

The Panel believes that the contribution of the [ ] a,c should be considered in determining the realistic in-plane shear capacity of the SC shield wall. The Panel notes that for SSE loads, WEC conservatively ignored the contribution of the [ ] a,c and used the in-plane shear design capacity of the shield building wall as [ ] a,c WEC's estimate of the expected in-plane shear capacity based on Japanese tests that includes the inner and outer steel plates is [ ] a,c WEC supported the Japanese test based in-plane shear capacity [ ] a,c The Panel is of the view that it will be prudent to reduce the estimated in-plane shear capacity to account for uncertainties. The Panel considers a reduction of the in-plane shear capacity to half [ ] a,c as reasonable to account for potential uncertainties. The computed in-plane shear force demand in the shield building wall due to SSE [ ] a,c is lower than the reduced in-plane shear capacity of [ ] a,c This consideration leads to a more realistic SSE design margin of [ ] a,c for in-plane shear force.

4.4.3 Conclusion

The panel concludes that, notwithstanding the results of its recommendation in Section 4.1.4, the certified shield building design meets the GDC- 2 requirements. On the basis of its assessment of Safety Concerns # 1 and 3 in Sections 4.1 and 4.3 respectively, the Panel further concludes that the AP1000 Shield Building is of sufficient strength and ductility to resist a review level earthquake (0.5g) that is larger in magnitude by 67% than the design basis SSE earthquake (Section 4.1.2.4).

In the Panel’s opinion, the design margin for the in-plane SSE shear force for the shield building wall is [ ][a,c] In the Panel’s expert judgment, this is a reasonably sufficient design margin to accommodate potential concerns that may not have been addressed explicitly. On this basis, the Panel believes that the certified AP1000 shield building wall design meets the GDC 2 requirement with respect to the SSE event.

5.0 Conclusions:

5.1 Safety Concern # 1 - The certified design of the AP 1000 shield building does not meet the NRC’s seismic margin requirements

The Panel concludes that the AP1000 SB does meet the NRC’s seismic margin requirements.

The Panel also concluded that: (a) The Westinghouse Electric Company (WEC) did not calculate the High Confidence Low Probability of Failure (HCLPF) values consistent with the shield building reanalysis results that were reported in Appendix L to the shield building design report, and recommends that the staff follow-up on the revised HCLPFs for shield building structures, and; (b) the HCLPF values reported in Table 19.55-1 of the DCD are at variance with the ones reported in Chapter 11 (pages 16-23) of the shield building design report, and recommends that the staff follow-up to resolve this discrepancy (Section 4.1.3).

5.2 Safety Concern # 2 - The NRC staff’s conclusion that the aircraft missile would not penetrate the AP1000 shield building wall is not logical.

The Panel concludes that the AP1000 Shield Building shell, constructed of steel-concrete (SC) modules, would not be perforated by the aircraft impact at room temperature (Section 4.2.3). This conclusion is based on three independent AIAs performed by WEC, the NRC Office of Research (RES), and the Panel's expert, Dr. Rashid. All three concluded that the AP1000 SB shell constructed of steel-concrete (SC) modules was not perforated by the aircraft impact at room temperature.

The Panel’s conclusions on temperature effects for the Shield Building AIA, the AIA for the Passive Cooling System (PCS) tank and benchmarking of computer models for AIA, are provided below:

(a) Temperature effects in the AIA evaluation of the Shield Building Wall – Although this issue was not specifically raised in the SOC, the Panel considered it important to address the issue. The Panel members were of different opinions on this issue:

Panel members [redacted] and Hackett consider that the WEC evaluation of this issue was appropriate and in accordance with the NEI AIA guidance regarding use of *best-estimate properties for the beyond design basis assessment (Section 4.2.3.5.1(a)).*

Panel member [redacted] *agrees with the Submitter's concern stated as, "the ability of the shield wall to resist aircraft impact under cold weather has not been substantiated and is doubtful." He recommends that the staff follow-up on this issue (Section 4.2.3.5.1(b)).*

(b) *The AIA for the PCS tank - Based on the evaluation in Section 4.2.4 of the report, the Panel understands that the structural integrity of the PCS tank and steel containment to withstand physical and shock effects due to aircraft impact was not inspected and verified by the NRC staff. Although the assessment was reviewed by the ACRS, the Committee did not conduct a quantitative verification of the WEC analyses. The Panel was informed by WEC regarding an overview of the AIA for the PCS tank that they performed, but we could not perform our own independent assessment due to insufficient resources. Therefore, the Panel concludes that that it cannot provide an independent opinion on the reasonable assurance of the validity of the WEC assessment of the PCS tank impact and the steel containment assessment for potential debris impacts. The Panel recommends that the staff follow-up on this issue and perform a review of the WEC assessments of these critical structures (Section 4.2.4).*

(c) *Benchmarking of computer models for AIA - the Panel concludes that it is not able to determine that WEC has met the intent of NEI 07-13 subsection 2.4.1(4) with regards to benchmarking of computer models for AIA.*

5.3 Safety Concern # 3 - The certified shield building wall does not possess sufficient strength and ductility to resist an earthquake and/or aircraft missile impact loading that is specified by the NRC (Section 4.3).

The Panel concludes that he AP1000 SB possesses both sufficient strength and ductility to be appropriately resistant to seismic excitation and aircraft impact per NRC requirements. In particular, the Panel finds that:

(a) *The Submitter's concern regarding additional demands due to "the irregular (Zigzag)" boundary condition, accidental torsion, and the P-Delta effects either have already been considered or do not result in higher design demands (Section 4.3.2 (a)).*

(b) *With regard to the Submitter's concern regarding the capacity of the Steel-Concrete (SC) module based on ACI Code 349, the Panel finds that WEC appropriately considered the pertinent ACI Code procedures, criteria and requirements. The Panel also finds that many of the ACI Code equations are applicable to the SC module element used in the AP1000 design (Section 4.3.2 (b) i and ii).*

(c) The Panel agrees with the Submitter that the ACI Code requires that ductility should be in every single structural component of the structure and that all members of the lateral force resisting system should be detailed for seismic ductile performance. (Section 4.3.2 (b) iii).

(d) The shield building SC modules as designed, will behave in a ductile manner even though some SC module elements with low shear-span-to-depth ratios failed in a brittle manner in laboratory tests. The Panel notes that RC beams with low shear-span-to-depth ratios also fail in a brittle manner (Section 4.3.2 (b) iv and Section 4.2.3.1).

(e) The Panel does not agree with the Submitter's concern regarding the use of a non-linear static push over analysis in lieu of the ACI code requirements. The Panel finds that the design basis of the shield building for the safe shutdown earthquake (SSE) is based on linear elastic response analysis and not on a non-linear static push over analysis. The push over analysis provided a qualitative understanding of the beyond design basis response and did not replace the ACI 349 design procedure or NRC requirements for the SSE. The use of static push over analysis for purposes other than the design is considered appropriate (Section 4.3.2 (b) v).

(f) The Panel agrees with the Submitter's concern that WEC did not consider a reduction in the ACI code capacity for in-plane shear strength due to interaction with out-of-plane shear forces. However, we conclude that this reduction in the in-plane shear strength is negligible and is not required by the ACI 349 Code (Section 4.3.2 (b) vi).

5.4 Safety Concern # 4 - The Certified shield building design does not meet General Design Criteria (GDC) 2 requirements.

On the basis of our conclusions regarding Safety concerns # 1 and 3 above, the Panel concludes that the AP1000 SB meets the requirements of General Design Criterion 2, "Design bases for protection against natural phenomena." (Section 4.4.2)

During the course of our review, the Panel made certain observations regarding the design, analysis and construction aspects of the shield building and the staff's technical and regulatory review processes. The Panel identified certain issues for the staff to follow-up. The observations and associated recommendations are detailed in Section 7.0 of the report. One of the observations deals with the shear reinforcing details at the connection of the PCS tank outer wall to the conical roof (Section 7.4.4). The Panel strongly recommends that this detail be modified. Another observation deals with the design and constructability of the tapered SC shield wall in the air-inlet region transitioning from 54 inch to 36 inch in thickness (Section 7.1). The Panel strongly recommends that the shield wall transition detail in the air inlet region be reviewed to ensure the tapered region is adequately designed, detailed, and is constructible.

The Panel also recommended that the staff follow up on the design and analysis of the PCS tank for design basis loads (Sections 7.4.1, 7.4.2, 7.4.3 and 7.4.5).

Prior to completion of the Panel's review, one of the experts retained by the Panel, Mr. Loring Wyllie, transmitted a letter to the Panel [a] highlighting and emphasizing two concerns he had previously identified in his report [b]. Due to the added emphasis, tone and contents of the letter, the Panel specifically referred these concerns to NRO via email on December 28, 2012 [c] for appropriate disposition.

Finally, the Panel made some observations and recommendations regarding the DPO process and will forward these under separate cover to the DPO Project Manager.

References

[a] "AP1000 Shield Building," Letter from Loring Wyllie, Jr. to Panel Member [redacted] December 13,

[b] "Expert Report of Selected Questions, AP1000 Enhanced Shield Building," Loring A. Wyllie, Jr., June, 2013

[c] Email from Panel Chair E. Hackett to NRO Director G. Tracy, December 27, 2013

6.0 Recommendations

The Panel made certain recommendations regarding the assessment of the shield building structural integrity in Section 4.0 and regarding the design, analyses, construction, and regulatory review process-related observations in Section 7.0 of the report. The following is a summary of the shield building structural integrity issues where the Panel made specific recommendations:

6.1 WEC did not re-evaluate the HCLPF values consistent with the shield building reanalysis results that are reported in Appendix L (Section 4.1.3).

6.2 The HCLPF values reported in Table 19.55-1 of the DCD are at variance with the ones reported in Chapter 11 (pages 16-23) of the shield building design report (Section 4.1.2.2).

6.3 Effect of cold temperatures on the shield building AIA – Panel member [redacted] recommends that the staff follow-up on this issues (Section 4.2.3.5.1(b)).

6.4 The Staff did not evaluate technical issue (c) in Safety Concern # 2 of the SOC and did not inspect the WEC PCS tank and the containment assessment for the physical and shock effects of the aircraft impact (Sections 4.2.4, 4.2.4.1, and 4.2.4.2).

7.0 Observations Regarding DPO-2012-002

During the course of our review of the DPO issues, the Panel and our experts made some observations regarding: (A) the analysis, design, and construction aspects of the shield building, and (B) the staff's technical and regulatory review process. These observations were not necessarily part of the SOC for the DPO. However, the Panel considered that they were important enough to forward for further consideration. The Panel recommends that the staff follow-up on the issues identified.

A. Technical Observations

7.1 Design of the SC Shield Wall Air Intake Region

The panel's expert, Mr. Wyllie, expressed a concern regarding the design of the shield wall in the air intake region beneath the tension ring where the shield wall tapers from a 54 in thick SC wall to a 36 in thick SC wall and contains a series of [ ] [a,c] sloping downward from inner edge of the steel plate wall to the outer edge. The inner steel plate of the SC wall is inclined to achieve the change in thickness from 54 inch above to 36 inch below. The air intake region and the tension ring above are both filled with self-consolidated concrete (SCC). There appears to be an apparent lack of structural continuity and strength across the air intake region particularly due to unsymmetrical loads on the roof of shield building from wind, seismic, or aircraft impact loads. Of particular concern is the constructability of design detail in the air intake region of the certified design.

Design of tie bars in the tapered air intake region is addressed on page H-90 of the shield building design report. A free body diagram of forces acting at the air inlet structure inner steel plate is shown in Figure H.3-13 (page H-91). The design forces in the transition (offset) area are illustrated as [ ] [a,c] in the inner steel plates. This is somewhat surprising because one would expect a substantial [ ] [a,c] in the top of the roof beam/concrete (like a fixed end negative moment) as the heavy roof steel beam and concrete slab is well connected to the tension ring. This would cause [ ] [a,c] forces in the bottom flange of the roof beam and in the inner face of the SC steel plate. Gravity loads from the shield building roof and water tank would also cause compression in the inner steel plate. The concern is not whether the inner steel plate is in tension or compression as it may be in either state of stress under certain loading conditions. The primary concern is that the horizontal component of force at both transition bends must be adequately resisted by tie bars or stiffeners. The design and construction of the horizontal tie bars or stiffeners is complicated by the sloping air intake pipes which cross the horizontal location.

The Panel notes that the staff accepted (in the FSER) the following attributes of the tie-bar design provided in the DCD.

(a) WEC does not require NRC approval for omitting any tie bars. “Tie bars and studs may be omitted in local areas due to design features and other obstructions”. (DCD Appendix 3H, Section 3H.5.6.1).
(b) The Panel could not find information to judge whether the magnitude of the assumed axial force in the design of tie-bars is reasonable. “The calculation assumed an axial force demand of [ ] a,c (FSER Section 3.8.4.1.1.3.7).
(c) The Panel does not believe that an allowance of two percent is sufficient and reasonable to account for uncertainty in the analysis results considering the complexity of the design and analysis of this local area. “Two percent of the value may be added to the design limit as an allowance for minor variances in analysis results” (DCD Appendix 3H, Table 3H.5-9, sheet 2c of 3).
(d) “Because of the amount of congestion in this area, constructability studies are being performed” (Staff’s FSER and WEC DCD).

The Panel therefore concludes that the constructability of the tie bar detail for the approved certified design is not yet established. Based on the observations (a) through (c) above, the Panel recommends that the staff should follow-up on this issue to ensure that the horizontal component of combined design basis force at both transition bends is adequately resisted by the tie bars and the air intake region is properly designed, detailed, and is constructible.

7.2 SC Wall Connections

The Panel’s expert, Mr. Wyllie, made an observation regarding the detail where the SC wall connects with the reinforced concrete wall as characterized below:

In this connection region, [ ] a,c This is an excellent detail to transfer high tension forces in the SC steel plates to the reinforced concrete wall below. However, if the steel plates of the SC wall are in substantial compression (especially under gravity plus seismic overturning compression) the design assumption [ ] a,c (See Design Report H.2.1.2, page H-66). It was not entirely clear how the compression in the steel plates will be transferred [ ] a,c to follow the load path suggested in the Design Report.

Based on his review of the Design Report, Mr. Wyllie believes these connections are adequate and that the design loading demands have adequate capacity. However, Mr. Wyllie also suggested some potential enhancements to the WEC design details in this region as follows:

This detail could be modified to also resist compression in the steel plates by adding a second nut to the [ ] a,c also adding a horizontal tie across the wall at the top of the bracket and by using a small lift of self-consolidating concrete so there will be sound concrete beneath the horizontal plate. This minor modification to WEC’s detail will allow for enhanced transfer of compressive forces from the SC wall to RC wall without the potential for damage to the concrete.

7.3 Auxiliary Building RC Roof Connection to the SC wall

The Panel's expert, Mr. Wyllie, made an observation regarding the detail of the connection of the Auxiliary Building roof slab to the SC wall of the Shield Building as follows:

Section 4.2.2 (page 4-23) of the Design Report discusses the connection of the Auxiliary Building roof slab to SC wall of the Shield Building. The RC roof slab has [ ] a,c Connection reinforcement, shown in Figure 4.1-6, (page 4-10) presumably [ ] a,c This is an acceptable detail, but a simpler and potentially better performing detail would be to provide holes in the outer plate of the SC wall and passing the roof slab dowel into the SC wall and welding it only to the inner plate. This would require a horizontal field joint in the SC wall plates slightly above to facilitate the field welding.

Regarding the shear lug detail shown in Figure 4.2-6 (page 4-25) - Following the calculations on page 4-24, each [ ] a,c This force gives a moment of [ ] a,c at the centerline of the ¾ inch SC wall plate, half being resisted each way. [ ] a,c This detail should be subjected to further review.

7.4 PCS Tank Design Considerations

7.4.1 Design basis operating temperature

The Panel noticed some inconsistencies in the DCD description of the design basis operating thermal load considered for the design of the PCS tank. The Panel observed that a differing description of the design temperature is provided in different sections of the DCD. For example, on page 6.2-99, Table 6.2.2-2, it states, "Design temperature of 125°F is specified for Passive Containment Cooling Water Storage Tank" that is different from the statement on page H-6, "Normal thermal loads for the passive containment cooling system (PCS) tank design are calculated based on the outside air temperature extremes specified for the safety-related design. The PCS tank is assumed to be at 40°F when the outside air temperature is -40°F. The water in the PCS tank is assumed to be at 70°F when the outside air temperature is postulated to be at 115°F."

The Panel had discussions with WEC to gain a better understanding of the technical basis for "assuming" the tank water temperature at +40°F when the outside air temperature is -40°F. Apparently this assumption is not conservative because WEC in response to the staff's June 20th, 2011 audit question had demonstrated that if the tank water temperature is at +50°F, the stresses in the tank wall will increase.

Based on a discussion with WEC, the Panel understands that the tank heaters are only activated to insure the water temperature does not fall below 40°F. However, the Panel did not understand the technical basis of the WEC assumption that the tank water temperature at +40°F is conservative given that tank water temperature above +40°F resulted in an increase of the tank wall stress and reduced the tank design margin. The Panel recommends that the staff should follow-up on this issue.

7.4.2 Consideration of uplift due to the vertical component of the SSE

The Panel noted that the peak ground acceleration (PGA) of the vertical component of the SSE seismic load (vertical acceleration response spectra in DCD: Pages 3G-72, 3G-73, and 3G-74 @ EL 327') exceeds [ ] [a,c] This indicates that a net vertical uplift load on the tank should be considered. The Panel did not find any discussion on this issue for the tank design.

Also, from the description of the hydrodynamic pressure on the PCS tank wall in Appendix H of the shield building design report, it is not clear whether the wall pressure due to the vertical component of the SSE has been computed and combined with the pressure due to the horizontal component of the SSE and whether the hydrostatic pressure is combined with the hydrodynamic pressure.

In a phone interview with the Panel, WEC clarified that their computer analysis of the PCS tank considered appropriate hydrodynamic pressure due to vertical component of the earthquake (uplift) and that hydrostatic pressure is combined with the hydrodynamic pressure due to horizontal and vertical components of the design basis earthquake. The Panel did not review or verify the WEC computer analyses.

The Panel recommends that the staff follow-up on this issue.

7.4.3 Stability of the compression ring

The Panel did not find any discussion of the stability of the PCS tank compression ring in the shield building design report or the DCD. When wind or seismic loads or any horizontal force is applied to this PCS tank, there will be an overturning force with more downward load on one side of the conical roof and less on the opposite side. It would seem that this would cause a twisting effect on the compression ring rotating it from its horizontal plane. What resists this twisting of the compression ring? The concrete walls above may help, but it would seem that very strong flexural continuity is needed across the air discharge area. It is envisioned that the PCS tank structure will rotating in more or less rigid body action causing rotation or twisting of the compression ring from its horizontal position. The only structure appearing to resist this rotation is a complex load path around the compression ring as it attempts to provide stability. From the information contained in the shield building design report, it was not clear how WEC modeled this area in their computer models. Apparently, WEC must have had a stable system as on Page H-172 they describe small vertical deflections of the conical roof beams, although they do not provide deflection data for the compression ring. In a phone interview with the Panel, WEC clarified that their computer analysis of the PCS tank includes the compression ring and believed it to be stable. The Panel did not review or verify the WEC computer analyses.

The Panel recommends that the staff follow-up on this issue.

7.4.4 Ability of the structural details to resist shear failure

The Panel noted that in the knuckle region, as shown in Figure 11.3-1 (page 11-26), the outer tank wall has [ ] [a,c] Then at the base there is a [ ] [a,c] which hooks down into the plane of the outer wall's outer curtain of vertical reinforcing bars. These [ ] [a,c] are critical to this knuckle joint to carry the shear from the base of the wall into the conical roof slab.

Also, at the base is a [ ] [a,c] which hooks down into the plane of the outer wall's outer curtain of vertical reinforcing bars. These [ ] [a,c] are intended to resist the radial shear forces at the bottom of the tank wall where the highest shear forces exist. The base of the wall where it meets the conical roof slab forms a corner that must resist internal water pressure. The Panel could find no precedent for the way the [ ] [a,c] are currently detailed. Precedents for good detailing of reinforcement at the corners of tanks (e.g., spent fuel pools) would have both the [ ] [a,c] hooking up (not down) into the plane of the outer wall's curtain of vertical reinforcing bars.

The implication of this observation can best be understood by drawing a diagonal crack beginning from the inside corner of the intersection of the tank wall and conical roof slab and extending downward to the outside face of the tank wall. The objective of the [ ] [a,c] is to tie together the two sections of concrete that are separated by the diagonal crack. Note that in the current detail both ends of the [ ] [a,c] lie in the same section, which does not enable the bars to tie the two sections together. Changing the detail so that both the [ ] [a,c] hook up (not down) into the plane of the outer wall's curtain of vertical reinforcing bars, would have each end of these bars anchored in a different section, which would tie the two sections together. This is how all corners of internally pressurized tanks are reinforced.

In a phone interview, WEC clarified that they have re-reviewed the detail and believe that the detail is adequately designed for the loading demands imposed on it. The Panel strongly disagrees with WEC and recommends that the detail be modified and staff follow-up on this issue.

7.4.5 Concrete underlying the tank in the Tank-roof slab connection

The Panel while reviewing the WEC shield building design report was concerned regarding the constructability of the PCS tank-roof connection particularly the potential of voids that could be formed in the concrete underlying the tank when the concrete is poured beneath the base of the tank. The Panel noticed a discrepancy in the information regarding as to how this concrete is to be placed. Section 6.2.2.1 (page 6-14) and Figure 6.2-15 (page 6-33) clearly show and state that the conical roof slab beneath the tank is placed [ ] [a,c] Then in Section 9.3 (page 9-5) and again in Section H2.2.2 and Figure H.2-14 (page H-76), it is stated that this concrete will be placed [ ] [a,c] The Panel realizes that SCC can be properly placed, but the Panel is concerned about potential trapped air at the bottom of this space causing concrete voids.

In a phone interview, WEC clarified to the Panel that the figure H.2-14 is the correct detail that is intended for construction and that a self-consolidating concrete (SCC) will be placed under the stainless steel tank liner.

WEC also stated to the Panel that they appreciated the Panel's caution regarding the potential voids in the concrete, and that they have discussed this issue with their constructor and are confident that they will [ ] [a,c]

The Panel recommends that the staff follow-up on this issue.

7.4.6 Seismic Margin and HCLPF

The Panel noted that the final design margin ratios for the PCS tank walls are summarized in Appendix L (Figure L.4-39 and Table L.4-5 (page L-75)) of the shield building design report. Based on the shield building reanalysis, the highest design [ ] [a,c] is reported in the [ ] [a,c] A HCLPF value of [ ] [a,c] calculated using the PCS tank wall design margin of [ ] [a,c] is reported in Section 11.3.3.2 of the shield building design report. However, the shield building reanalysis in Appendix L shows the PCS tank wall design margin is reduced from [ ] [a,c] The Panel could not locate the HCLPF calculation based on this design margin of [ ] [a,c]

The Panel recommends that the staff follow-up on this issue.

7.5 Staff's Seismic Evaluation in FSER Section 3.7

The Panel observed that the staff in FSER Section 3.7.2 on seismic evaluation, did not provide a description of its evaluation of the following specific SRP acceptance criteria 3.7.2.II.9 through 14, that are required to meet the relevant requirements of the NRC's regulations. More specifically, the Panel did not find FSER sections 3.7.2.9 through 3.7.2.14 that correspond to SRP acceptance criteria 3.7.2.II.9 through 14. The Panel did not find any reasons for not providing any evaluation to these criteria or any reference to Sections where such evaluations are provided.

3.7.2. II. 9. Effects of Parameter Variations on Floor Response Spectra.
3.7.2. II.10. Use of Equivalent Vertical Static Factors.
3.7.2. II.11. Methods Used to Account for Torsional Effects.
3.7.2. II.12. Comparison of Responses
3.7.2. II.13. Analysis Procedure for Damping.
3.7.2. II.14. Determination of Seismic Overturning Moments and Sliding Forces

The Panel recommends that the staff follow-up on this issue to determine whether the seismic analyses of the certified AP1000 design have been evaluated and documented in the FSER for the SRP acceptance criteria 3.7.2.II.9 through 14.

B. Technical and Regulatory Review Process Observations

1. The Panel considers that there is an opportunity for improvement in the overall safety review process. In the current review process, reliance is placed to a large extent on the reviewer's knowledge and the work is not generally verified by another knowledgeable reviewer. Currently, there is no requirement for a peer review of the reviewer's work. Under 10CFR 50, Appendix B quality assurance program, licensees are required to independently verify all design and analysis calculations. Generally, in the current staff review process, the first line supervisor (e.g., Branch Chief) represents and performs the management function and does not necessarily have an in-depth knowledge of complex technical matters and issues. The Panel recommends that a peer review of the staff's safety reviews work product should be performed concurrently by another reviewer. The Panel believes that such a peer review would significantly improve the overall quality of NRC staff's safety reviews, help to minimize potential non-concurrences and DPO filings, and result in the enhancement of the overall efficiency and effectiveness of safety reviews.

2. The Panel observed an opportunity for improvement in the aircraft impact assessment (AIA) reporting and the staff's inspection requirements. Subsequent to the staff's inspection of the AIA, WEC performed a supplemental AIA of critical structures (PCS tank and steel containment) but apparently failed to report it to the staff. The AIA of critical structures is not described or documented and WEC confirmed to the Panel that it was not verified by the staff's inspection.

3. The Panel observed an opportunity for improvement in the control of reporting design information. WEC reported HCLPF values in two separate documents (DCD and the shield building design report) that were in conflict with each other. WEC committed to take corrective action.

4. The Panel observed an opportunity for improvement in the staff's review of technical information. WEC did not document the HCLPF values for the final combined design load. Apparently WEC assumed that HCLPF values are independent of the ambient thermal condition but the WEC did not find any technical basis of the assumption to the Panel.

8.0 Appendices

A. Appendix A - Statement of Concerns

B. Appendix B – Records and Documents Reviewed by the DPO Panel

C. Appendix C – Stakeholders Interviewed by the DPO Panel

D. Appendix D – Summary of Important Email Communications

E. Appendix E – Crosswalk of DPO Issues with Important Email Communications

Appendix A – Statement of Concerns

Statement of Concerns (SOC)
By
AD HOC Review Panel on
Differing Professional Opinion (DPO) on the AP1000 Shield Building
DPO-2012-002

The purpose of the Statement of Concerns (SOC) is to document the DPO Panel's ('Panel') understanding of the DPO Submitter's ('Submitter) concerns and the Panel's review scope for DPO-2012-002. The Submitter has characterized the certified AP1000 shield building as being unsafe on the basis of the four primary concerns identified below. These concerns are identical to those that the Submitter identified in the original NRC Form 680, which documents DPO-2012-002. The Panel reviewed the Submitter's detailed discussion of each of the concerns in the DPO submittal and had meetings with the Submitter to clarify our understanding of the technical issues. The Panel's understanding of the safety concerns and identification of technical issues underlying each concern is provided below. Additional discussion of specific technical issues is included in Items 1-12 of the Attachment provided by the Submitter.

<u>Safety Concern and Technical Issues</u>

<u>Safety Concern # 1</u> -The certified design of the AP-100 shield building does not meet the NRC's seismic margin requirement.

Technical Issue:

The in-plane shear demand exceeds the ultimate in-plane shear strength (capacity) and therefore, high confidence of low probability of failure (HCLPF) is significantly lower than the review level earthquake (RLE) of 0.5g. (Item 7).* Therefore, the AP-1000 shield building does not meet the NRC's seismic margin requirement.

(Note: * refers to the item number in the Attachment)

<u>Safety Concern # 2</u> – The NRC staff's conclusion that the aircraft missile would not penetrate through (perforate) the AP1000 shield building wall is not logical.

Technical Issues:

(a) Out-of-plane shear tests showed that the shield building wall behaved in a non-ductile manner and therefore has insufficient punching shear strength and ductility to resist the aircraft missile. (Item 8)*

(b) The NRC's conclusion that the aircraft missile would not penetrate through (perforate) the shield building contradicts the aircraft missile impact results for the Economic Simplified Boiling Water Reactor (ESBWR) and the South Texas Project Nuclear Operating Company (STPNOC) amended Advanced Boiling Water Reactor (ABWR) designs that were submitted for design certification.

(c) The connection between the shield building roof and the water storage tank is not adequate. Failure of the connection may lead to an impact of the water storage tank onto the containment and the potential compromise of the primary system. (Items 10 and 12)*

(d) The structural wall may behave in a non-ductile manner under impact loading. (Item 9)*

<u>Safety Concern # 3</u> -The certified shield building wall does not possess sufficient strength and ductility to resist an earthquake and/or aircraft missile impact loading, that is specified by the NRC.

Technical Issues:

(c) Seismic demand was under predicted and the actual demand required for the design will be higher due to WEC <u>not</u> having considered the following: (Item 11)*

(iv) The inherent torsion due to the irregular (Zigzag) boundary conditions.

(v) The demand~~s~~ from accidental torsion required by the American Concrete Institute (ACI) Code and the Standard Review Plan (SRP), NUREG-0800.

(vi) The analysis for the ACI Code requires consideration of second order P-Delta effects. The water tank resting on top of the shield building, containing approximately 7 million pounds of water, will add to the P-Delta effect.

(d) The Steel-Concrete (SC) module Capacity is determined incorrectly due to the following:

(vii) The ACI Code design equations are not applicable to the S-C module elements used in the AP1000. (Item1)*

(viii) WEC ignored the ACI Code's procedures, criteria and requirements, which were endorsed by the NRC, for the design and review of concrete buildings. (Item 2)*

(ix) The NRC staff misinterpreted the ACI Code's ductility requirements for concrete elements for seismic design as being "no requirement." That is "The Code does not specify a ductility level nor does it specify that ductility should be in every single structural component of the structure." (Item 3)*

(x) The NRC staff accepted the SC module element even though when laboratory tested, it failed in a brittle manner and did not meet the acceptance criteria established jointly by the NRC and WEC.

(xi) The NRC staff accepted a non-linear static push over analysis *in lieu* of the ACI code required testing as a basis for the shield building SC module design. Push over analysis is inappropriate and inadequate. NRC improperly accepted push over analysis neglecting test results. The pushover analysis method is prohibited by building codes for use on an irregular, tall, or safety important building such as the shield building. Therefore, the NRC failed to understand that the nonlinear static pushover analysis is not applicable to the AP1000 shield building. The Nuclear Installation Inspectorate (NII) of the United Kingdom concluded that the pushover analysis is not applicable to the shield building, and both NII and Brookhaven National Laboratory (BNL) discovered many problems with the WEC analysis and results. (Items 4, 5 and 6)*

(xii) WEC did not consider the ACI code capacity for in-plane shear strength reduction due to interaction with out-of-plane shear forces.

<u>Safety Concern # 4</u> – The Certified shield building design does not meet General Design Criteria (GDC) 2 requirements.

The concern that the shield building design does not meet the GDC 2 requirements is embodied in the three safety concerns and technical issues identified above, and in the overall concern that the AP1000 shield building has the following three major differences with the shield buildings in the operating plants. (Item 12)*

(a) The shield building is joined together by two different types of construction (i.e., Reinforced concrete and SC module). The “constitutive laws” are available for reinforced concrete (RC) modules (elements), however they are lacking for the SC module.

(b) The two different types of construction are joined in a zigzag shape at boundaries in both circumferential and meridional directions and such irregular boundaries may create non-symmetric seismic responses for the two portions of the shield building.

(c) A water tank, containing approximately 7 million pounds of water, rests on top of the AP1000 shield building roof. There is a concern that under seismic or impact loading, that failure of the water tank structure could cause consequent failure of the containment and compromise of the primary system. (Items 10 and 12)*.

(d) With the understanding of the above three major differences between the existing reinforced concrete (RC) shield building in operating plants and the certified AP1000 shield building, there is a concern that the certified AP-100 shield building does not meet the GDC-2 requirements.

ATTACHMENT – Additional Detail Provided by the Submitter

1. *The AP1000 cylindrical shield building wall design does not meet ACI building codes*

ACI building Codes have been endorsed by the NRC in the Standard Review Plan and Regulatory Guides for the design and evaluation of seismic Category I structures, including shield buildings, and is the licensing basis for the AP1000 shield building, as stated in the AP1000 DCD. ACI codes require that the cylindrical shield building wall to possess sufficient strength, ductility, and energy dissipation/absorption capability under design loads, including earthquakes and aircraft missile impact. ACI codes provide procedures and methods for analysis and design for concrete structures to comply with the codes. For the cylindrical shield building wall, the procedure is (1) to size the thickness of the cylindrical shell and provide the amount of reinforcement at all locations in the shell (the particular location is called an "element"), and (2) to perform analyses for the entire shield building subjected to design loads and demonstrate that all elements in the shield building wall meet the criteria of strength, ductility, and energy dissipation/absorption capability specified by the codes *(Report Sections 4.2.3.1, 4.3.2(b))***.**

For ordinary reinforced concrete elements, steel reinforcing bars are fully embedded in and surrounded by concrete, and they act as a unit. Although to achieve the proper sizing of elements is a trial and error procedure between (1) and (2) as stated above, the method to achieve that proper sizing of the thickness and the amount of reinforcement is straightforward, and available in the codes. This is because constitutive laws of ordinary reinforced concrete elements are known after decades of physical testing and analytical research. Constitutive laws for a reinforced concrete element are the mathematical relationship between the internal strength, ductility, and energy dissipation/absorption capability of the element under external applied loads. These constitutive laws are required to form the basis for analysis methods or computer codes for computing the behavior of a structure (the shield building in this case) under loads as stated in the procedure (2) in the previous paragraph. However, the element of the AP1000 shield building wall is built with elements that consist of two steel faceplates with concrete filled between them, which is called SC concrete elements or modules, which are not an ordinary reinforced concrete element and constitutive laws have not been established for the SC concrete element due to the lack of testing data. Under such a circumstance, ACI Code requires a testing route to justify the acceptance of such a structure. The NRC, assisted by two world leading authorities in concrete structure design for earthquakes, and the WEC, jointly established the acceptability criterion for the proposed SC concrete elements. The criterion is that the SC concrete element should possess a shear ductility ratio of three in both the radial (out-of-plane) and circumferential (in-plane) directions and verified by testing *(Report Sections 4.2.3.5.3, 4.3.2 (b) iii).*

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The out-of-plane shear test for the element, with tie-bars between the faceplates [ ] [a,c] failed in a brittle manner with ductility ratio near one. ACI code prohibits the use of brittle concrete elements. However, the NRC staff accepted the brittle concrete element against the ACI code and the acceptance criterion that was jointly established by the NRC and WEC *(Report Section 4.3.2(b) iv)).*

2. *The staff's acceptance of the brittle SC concrete element for the shield building wall is not only against its own acceptance criterion that was agreed upon between the staff and WEC but also provides no technical reasons*

   The staff accepted the brittle SC concrete element for use in the AP1000 shield building wall is against its own acceptance criterion. Under the strong protest from the Submitter and the two NRC consultants, the staff did not offer any satisfactory technical reasons for its action *(Report Sections 4.2.3.1, 4.3.2(b) iv, 4.3.2(b) v)).*

3. *The staff's acceptance of the brittle SC concrete element for the shield building wall is inconsistent with material science, testing data for seismic and impact loadings, and structural engineering principles, for the wall to resist missile impact and earthquake loadings*

   Material science indicates that brittle material does not resist cyclic and impact loadings well, and testing data in the laboratory have confirmed that. Structural engineering principles indicate that brittle elements, members, and connections between members, are the causes for structural failure and collapse, and field structural failure and collapse data due to earthquake and impact have confirmed that. Therefore, ACI building codes prohibits the use of brittle elements, members, and connections for concrete structures, and require the use of ductile elements, members, and connections. The code requires the design ductility value to be commensurate with and proportioning to the required seismic intensity for, and the importance of, the structure. However, the staff has taken a position to accept the brittle element for the AP1000 shield building wall to resist missile impact and earthquake loadings against the knowledge of material science, testing data for seismic and impact loadings, structural engineering principles *(Report Section 4.2, 4.3.2(b) iii)).*

4. *The staff's acceptance of the non-linear static push-over analysis for the adequacy of the shield building under earthquake loadings violates building codes' requirements*

   WEC used a non-linear static push-over computer code instead of dynamic analysis, which is required by the NRC Standard Review Plan and Regulatory Guides, as the demonstration of the adequacy of the shield building design under earthquake loading, and the NRC staff accepted the analysis and used the analysis results as the justification for the safe design of the AP1000 shield building. Research results indicate that, and building codes allow that, the non-linear static push-over analysis method is only applicable to a building structure that comply with the following three conditions simultaneously: (1) non-essential structures, (2) with a regular geometry or shape, and (3) , and low-rise structures about 40 feet in height. However, the AP1000 shield building is an essential building, with irregular restraining or supporting conditions, and has a height of about [ ] a,c and does not comply with anyone of the three conditions. Therefore, the non-linear static push-over analysis or computer code is not applicable to the shield building. The staff's acceptance is based on unsound technical basis *(Report Section 4.3.2(b)v).*

5. *The use of non-linear static push-over computer analysis by WEC for the seismic analysis for the shield building violates dynamic analysis principles and is inconsistent with the requirements of the standard review plan and regulatory guides, and thus the staff's acceptance of the WEC non- linear static push-over computer analysis and its results to justify the safety adequacy for the AP1000 shield building has no technical basis*

   The static push-over analysis captures only the first mode (or mode shape) responses of a cantilever column, such as the deflection of, or forces in, the shield building wall, under dynamic loadings, such as seismic or impact. Only dynamic analysis can capture the response from all of the different modes. Therefore, the NRC requires dynamic analysis for seismic Category I structures under dynamic ladings, as stated in the Standard Review Plan and Regulatory Guides. The staff's justification for its acceptance of the non-linear static push-over computer code is "The push-over method is an accepted industry practice for estimating the limit state (i.e., collapse load) and **corresponding modes** (emphasis added) of failure of a structure due to seismic loading…." The staff should recognize that only the first mode is considered in the non-linear static push-over computer analysis and all other modes are neglected. Therefore, the staff's acceptance is contradictory with the NRC requirement and the earthquake/dynamic engineering principles. The staff's justification is technically incorrect *(Report Section 4.3.2(b)v).*

6. *The non-linear static push-over computer code used by WEC for seismic analysis for the shield building with SC concrete elements is not valid because constitutive laws are not available for these elements due to the lack of testing data, and the staff's acceptance of the computer code and the justification of its acceptance are baseless*

   Structural analysis computer codes require information of constitutive laws for elements of the structure, and without them the analysis cannot be performed and the computer code cannot be written. Constitutive laws for elements of ordinary reinforced concrete structures are available due to decades of analytical research and calibrations with physical testing data. However, constitutive laws for SC concrete elements are not available or insufficient even in the linear range and certainly unavailable or unreliable to extend into non-linear range for non-linear analysis. WEC had to assume some types of constitutive laws, which are not obtained from or validated by test, for SC concrete elements of the shield building wall in its non- linear static push-over computer code, and thus the analysis results can be far from valid or reliable or even meaningful. The staff's acceptance of such unreliable or invalid analysis results can be dangerous with respect to safety. The staff's justification for its acceptance of the non-linear static push-over computer code is "The push-over method is an accepted industry practice for estimating the limit state (i.e., collapse load) and corresponding modes of failure of a structure due to seismic loading…." It is not possible to reach a limit state or obtain collapse load for the shield building if constitutive laws of its elements are not available or insufficient or unreliable. Therefore, the staff's justification for its acceptance of the WEC non-linear static push-over computer code and analysis results is baseless *(Report Section 4.3.2(b)v).*

7. *The shield building wall does not meet the NRC seismic margin assessment regulation due to insufficient thickness of concrete in the wall to resist in-plane shear generated by earthquakes*

   The Submitter demonstrated that the certified AP 1000 shield building does not meet the NRC Seismic Margin Assessment regulation because the in-plane shear stress exceeded the ultimate in-plane shear strength for the entire height of the wall under the Review Level Earthquake – evidence of insufficient thickness of concrete in the wall *(Report Section 4.1).*

8. *The shield building wall does not meet the NRC aircraft missile impact assessment regulation due to insufficient thickness of concrete in the wall to resist punching shear generated by the aircraft missile*

   The punching shear resistance of a shield building wall is proportional to the wall's shear strength, ductility, and energy absorption capability. The three feet six inches thick ordinary concrete wall of the reactor building in ESBWR and approximate four feet thick ordinary concrete wall in the reactor building near the spent fuel pool area in STP were penetrated through by aircraft missiles through non-linear computer codes specifically developed for the aircraft missile impact analysis. It is logical to conclude that the three feet thick AP1000 shield building wall would be penetrated through by aircraft missiles because test has shown that the SC concrete wall has less strength, ductility, and energy dissipation/absorption capability than that of a companion ordinary reinforced concrete wall - evidence of insufficient thickness of concrete in the Wall. It is not logical for the staff to conclude that the three feet thick AP1000 shield building wall would not be penetrated through by aircraft missiles *(Report Section 4.2).*

9. *The shattering effect of the brittle SC concrete elements in the AP1000 shield building wall under aircraft missile impact has not been investigated by WEC and the staff*

   A brittle structural wall may shatter into pieces when it is struck by impact loading. The SC concrete wall in the AP1000 shield building is brittle. The shattering effect due to aircraft missile impact on the wall has not been investigated by the WEC and the staff *(Report Sections 4.2.3. 4.2.4 and 4.2.5).*

10. *The connection between the water tank and the shield building roof does not appear to possess sufficient rotational rigidity to restraint the tank from being excessive rotation when the tank is struck by aircraft missiles and the tank will cause damage to the roof due to impact and fall through the roof and crash the steel containment and potentially damage the fuel and create an unintended accident with uncontrolled release of radiation.*

    The heavy water tank is sitting on top of the shield building roof and is supported by and connected to the roof. The roof does not provide as much rotational rigidity to the water tank compared to the condition if the tank is supported on the ground. Therefore, the connection between the water tank and the roof may not possess sufficient rotational rigidity to restraint the tank from being excessive rotation when it is struck by the aircraft missile. This excessive rotation may cause failure of the connection between the tank and the roof and the impact of the tank to the roof may cause damage to the roof and the tank to fall through the roof and crash the steel containment and potentially damage the fuel and create an unintended accident with uncontrolled release of radiation (Report Sections 4.2.4.1 and 4.2.4.2).

11. *The following analyses were not performed in obtaining the demand of forces or stresses in the shield building wall for earthquake loading: (1) forces or stresses generated due to the irregular shape of the restraining (supporting) system between the steel plate portion and the ordinary reinforced concrete portion of the shield building wall both in the circumferential and meridional directions, (2) accidental torsion effect, and (3) second order P-Delta effect, and the lack of those analyses resulted in less calculated demand of forces than the actual forces acting on the shield building wall due to earthquake loading*

    Analysis, testing, and building failure data after earthquakes have indicated that irregular supporting system would generate torsional forces or stresses in the structure compared to that if the supporting system is symmetrical and not irregular. Therefore, building codes discourage the use of irregular structures and specify penalties for the design proportionally to the degree of irregularities of the structure. For the AP1000 shield building, the portion of the SC shield building that contains SC concrete elements, is irregularly restrained or supported by the ordinary reinforced concrete portion of the shield building both in the circumferential and meridional directions. These irregularities were not well captured in the mathematical model, and their effect on forces or stresses in the shield building wall is not well unaccounted for.

    The as built shield building wall will deviate from a perfect cylindrical shell as assumed in the mathematical model, and this condition of an imperfect cylindrical shell creates accidental torsion during earthquakes. Concrete strength may vary from one batch to the other, and concrete density may also vary from one location to the other, and thus create accidental torsion in the shield building during earthquakes. Building codes and the NRC Standard Review Plan require accidental torsion analysis to be performed and the effect be accounted for. However, WEC did not perform such an analysis.

The first order linear elastic analysis assumed that the mathematical model of the shied building does not deform and therefore the forces resulting from the analysis does not account for the deformed shape of the shield building during earthquakes. ACI code requires a second-order P-Delta analysis to account for the additional forces generated due to the deformed shape of the shield building. Due to the heavy vertical weight of the water tank on top of the roof, the vertical load (P) is considerably larger on the shield building wall compared to that of other structures that do not have a heavy tank on their top, and the forces from the P-Delta effect could by significant. However, WEC did not perform the second order P-Delta analysis, and the forces from the P-Delta effect are not accounted for *(Report Section 4.3).*

12. *Overall high level assessment on the certified AP1000 shield building*

There are three major distinctions of the AP1000 shield building compared with existing shield buildings in the operating plants. First, all existing shield buildings are ordinary reinforced concrete structures, and are designed and constructed in accordance with ACI codes. However, the AP1000 shield building is joined together by two different types of components or sub-structures, with different concrete elements. One sub-structure is consisted of SC concrete elements. The other sub-structure is consisted of ordinary reinforced concrete elements. Second, existing shield buildings are continues reinforced concrete construction, but the AP1000 shield building is formed by connecting the two sub-structures with zigzag shapes at boundaries in both circumferential and meridinol directions. Third, no water tank on top of existing shield buildings in operating plants, but a heavy water tank is sitting on top of the AP1000 shield building roof.

The first distinction is the most important one with respect to the adequacy and reliability of design. Ordinary reinforced concrete building structures, including shield buildings, have a long successful history – a testimony to the reliable design and construction methods that were the results from decades of research as specified in the ACI codes. However, there is no building structure that was designed and built with the SC concrete elements, as proposed in the AP1000 shield building, in the United States, and there is no reliable analysis and design method for the SC concrete elements. Therefore, the adequacy and reliability of the design and construction of the AP1000 shield building are in question at the fundamental element level. Therefore, the adequacy and reliability of the AP1000 shield building are definitely less than that of the existing shield buildings, and the question is what is the level of adequacy and reliability that is needed for the design and construction of the AP1000 shield building.

The tie-bar that is welded between the steel faceplates serves two purposes: (1) to make the SC concrete element acting as a unit, and (2) to [ ] a,c

[

] a,c The brittle failure of the shear test at Purdue University demonstrates the validity of design philosophy and requirements of the ACI code, and of material science. Using brittle tie-bars to build the SC concrete element, the element will naturally be brittle. The energy dissipation capability of the companion ordinary reinforced concrete element in the test is about [
] a,c that of the SC concrete element for resisting seismic loads. Brittle tie-bars make brittle SC concrete elements. Brittle SC concrete element has small strength, ductility, and energy dissipation/absorption to resist shear force generated by earthquakes and punching shear by missiles. Stacking up brittle SC concrete elements into the SC portion of the AP1000 shield building, and that portion of the shield building will naturally be brittle as well. It is not logical to conclude that the 3 feet thick SC wall will not be penetrated through (perforated) by aircraft missiles while the 4 feet thick ordinary reinforced concrete wall in the STP project did, and the test data indicated that the SC concrete element has less strength and energy absorption capability than that of the companion reinforced concrete element. More importantly, while it is possible with some confidence that the non-linear behavior of the reinforced concrete wall, when it is struck by seismic or impact forces, can be numerically estimated because of the availability of constitutive laws, it is almost impossible or much less possible to estimate the SC wall's behavior due to the lack of information on constitutive laws for the SC concrete elements**.** This reliability issue should be considered by the NRC in its evaluation of the AP1000 shield building.

The second distinction is a design consideration between good and not good. The continuity of reinforced concrete structures of existing shield buildings provides continued and smooth force transfer from the roof of the building to its foundation, and does not create inherent torsion for the building during earthquakes, which is a good design. However, cutting the AP1000 shield building into two irregular portions with different types of concrete elements, and then connect them together, do not provide smooth force transfer from the top to bottom of the shield building and create inherent torsion for the building, which is not a good design. Failures or collapse of structures during earthquakes were mostly caused by bad design and construction.

The third distinction is both design and safety concerns. Placing a heavy water tank on top of the AP1000 shield building roof is not a good design consideration for a building that subjected to earthquakes, because the building behaves as an inverted pendulum which is an unfavorable geometry for seismic design and would generate more forces in the AP1000 shield building wall than that of existing shield buildings that do not have water tank on the roof. This distinction could be the main reason while a three feet thick concrete wall would be sufficient to resist earthquake forces for existing shield building but insufficient for the AP1000 shield building with the water tank on the roof.

Placing a heavy water tank on top of the AP1000 shield building roof creates a safety concern. This safety concern is that an unintended accident and uncontrolled release of radiation could result if the water tank falls through the roof and that would likely crash the steel containment and potentially damage the fuels. Therefore, to achieve a high confidence in design and construction to prevent the possibility of the water tank from falling through the roof is important and necessary. However, it is difficult for the certified AP1000 shield building to achieve that high confidence in design while using (1) the SC concrete element that has no reliable or valid constitutive laws, (2) brittle SC concrete element, and (3) two different types of concrete elements connected at zigzag boundaries to form the entire shield building, because these three items offer no or low reliability and thus no or low confidence for the AP1000 shield building analysis and design. *(Report Sections 4.2, 4.3, 4.4)*

Appendix B – Records and Documents Reviewed by the DPO Panel

1. WEC AP 1000 Design Control Document, Revision 16, May 26, 2007 (ML071580593)

2. Final Safety Evaluation Report Related to Certification of the AP 1000 Standard Plant Design, October 17, 2008 (ML112091879)

3. Design Report for the AP 1000 Enhanced Shield Building, June 15, 2011 (ML111950098)

4. Non-Concurrence – Safety Evaluation Report for AP 1000 Shield Building (ML103020207)

5. Differing Professional Opinion 2012-002 – Certified AP1000 Shield Building Unsafe, July 18, 2012 (ML12200A236)

6. Staff Requirements Memorandum (SRM) to SECY 93-087, "Policy, Technical, and Licensing Issues Pertaining to Evolutionary and Advanced Light-Water Reactor (ALWR) Designs," July 21, 1993 (ML003760768)

7. Differing Professional Opinion Regulatory Guide 1.217 – Guidance for the Assessment of Beyond-Design-Basis Aircraft Impacts, August 2011 (ML092900004)

8. Report on the Final Safety Evaluation Report Associated with the Amendment to the AP1000 Design Control Document, December 13, 2010 (ML103410351)

9. Letter to WEC from Dave Matthews to Rob Sisk regarding AP1000 Shield Building Design, October 15, 2009 (ML092320205)

10. Division of Engineering Evaluation of the Design for the AP1000, Revision 17, Enhanced Shield Building, Office of Nuclear Regulatory Research, October 29, 2010, (ML103080105)

11. Non-Concurrence – Safety Evaluation Report for AP1000 Shield Building, November 4, 2010, (ML103020207)

12. Wyllie, L, "Expert Report of Selected Questions AP 1000 Enhanced Shield Building," June, 2013

13. Rashid, J., "Expert Opinion on AP 1000 Shield Building Aircraft Impact Assessment," June, 2013

14. NEI 07-13 Revision 8P, “Methodology for Performing Aircraft Impact Assessments for New Plant Designs,” April 30, 2011 (ML111440006)

15. IR 05200006-10-203 and Notice of Violation on 09/27/2010 -10/01/2010, WEC Electric Co., AP1000 Pressurized Water Reactor Design Aircraft Impact Assessment Inspection, October 28, 2010 (ML102980583)

16. Reply to Notice of Violation Cited in IR 05200006/2010-203, November 12, 2010 (ML103210409)

17. ACRS Letter, “Draft Final Regulatory Guide 1.217, ‘Guidance for the Assessment of Beyond-Design-Basis Aircraft Impacts”, February 18, 2010 (ML101170344)

18. Official Transcript of Proceedings, Nuclear Regulatory Commission, Advisory Committee on Reactor Safeguards, WEC AP1000 DCD, August 16, 2011 (ML11242A164)

19. WEC-NRC Meeting on AP1000 Shield Building, May 17, 2011, Presentation Part 1 of 2, May 26, 2011 (ML111430803)

20. WEC-NRC Meeting on AP1000, May 17, 2011, Presentation Part 2 of 2, May 26, 2011 (ML111430602)

21. Response to Audit Questions - Shield Building Audit from June 20-24, 2011, dated June 27, 2011 (ML11181A011)

22. Proprietary Enclosure to Response to Audit Questions – Shield Building Audit from June 20-24, 2011, dated June 27, 2011 (ML11181A01)

23. Ozaki, M., S.Akitab, H.Osuga, T. Nakayamad, and N. Adachi. Study on Steel Plate Reinforced Concrete Panels Subjected to Cyclic In-Plane Shear, Nuclear Engineering and Design, 228, 1-3, pp. 225-244, 2004.

24. Mullapudi, TRS, P. Summers, I.H Moon, "Impact Analysis of Steel Plate Concrete Wall, "ASCE Structures Congress 2012.

25. Wright J.K. and M.A. Sozen, "Strength Decay of RC Columns under Shear Reversals," ASCE Journal of the Structural Division, May 1975.

26. Winter, George, and Arthur Nilson, “Design of Concrete Structures," McGraw-Hill, Ninth Edition, 1979.

27. Rolfe, S.T. and J.M. Barsom, "Fracture and Fatigue Control of Structures," Prentice-Hall, Inc., 1977

28. National Oceanic and Atmospheric Administration (NOAA) website www.NOAA.gov – United States State by State year round average temperatures

29. Rolfe, S.T., ”Fracture and Fatigue Control in Structures”, Engineering Journal, American Institute of Steel Construction, First Quarter/1977.

30. Vinson, J.R. and T.W. Chou, "Composite Materials and Their Use in Structures," Halsted Press, John Wiley and Sons, 1975.

Appendix C – Stakeholders Interviewed by the DPO Panel

John Ma, Senior Structural Engineer, NRO, DPO Submitter

Thomas Bergman, Director, Division of Engineering, NRO

Brett Tegeler, Senior Structural Engineer, NRO

Jose' Peres, Research Engineer, RES

Lupe' Arguello, Research Engineer, Sandia National Laboratory

Malcolm Patterson, Senior Structural Engineer, NRO

Doug Ammerman, Sandia National Laboratory

William Shack, Former Member, ACRS

Joe Rashid, ANATECH Corporation

Loring Wyllie, Degenkolb Engineers

Todd Baker, Fellow Engineer, WEC Electric Corporation

Lee Tunon-Sanjur, Civil Structures Consulting Engineer, WEC Electric Corporation

Paul Russ, Director, U.S. Licensing and Regulatory Support, WEC Electric Corporation

Appendix D – Summary of Important Email Communications

Date From/To	Subject	Disposition
1) 10/18/2012 Ma to Panel (1:47 pm)	Summarizes DPO concerns and forwards a New York Times article on the concerns and an email exchange with NRO staff and management.	DPO concerns were incorporated in the SOC and are addressed in the Panel report. The New York times article summarizes the previous non-concurrence at a high level.
2) 10/18/2012 Ma to Pane (5:06 pm)	Forwards cleaned-up version of 1 above including attachments.	Subsumed by final SOC.
3) 10/18/12 Ma to Panel (5:13 pm)	Forwards copy of the Submitter's 06/29/12 paper on the SB. Forwards copy of paper by F. Vecchio on ductility in shear response of SB. Forwards copy of comments by Hsu, T., and Mo, Y. on safety of SB.	Subsumed by final SOC. Technical information noted by the Panel.
4) 10/18/2012 Ma to Pane (6:35 pm)	Forwards BNL evaluation of LSDYNA analyses by WEC and technical information and internal NRO email correspondence.	Technical information noted by the Panel.
5) 10/23/2012 Ma to Panel	Forwards evaluation of SB by UK Nuclear Installations Inspectorate	Technical information noted by the Panel.
6) 10/25/2012 Ma to Panel	Forwards itemized list of DPO concerns.	Subsumed by final SOC and addressed in Panel report
7) 10/25/2012 Ma to Hackett and [redacted]	Forwards B. Harvey email announcing FR notice soliciting public comments on implementation of RG 1.221 on design basis hurricane missiles	For Information and Panel awareness.
8) 10/25/2012 Ma to Panel	Informs Panel where they can find certain information regarding wind generated missiles and aircraft impact	For Information and Panel awareness.

9) 11/07/2012 Ma to Hackett	Forwards review of proposed SOC and suggest modifications.	Subsumed by final SOC.
10) 11/12/2012 Ma to Hackett	Forwards high-level SOC statement. Reiterates contentions that building blocks for shield building are brittle, that static push-over analysis is not appropriate, and that the staff improperly released WEC from performing required analyses.	Subsumed by final SOC. Technical information noted by the Panel.
11) 11/26/2012 Ma to Hackett	Forwards Powerpoint presentation on good engineering practice.	For Information and Panel awareness.
12) 11/27/2012 Ma to Panel	Forwards WENRA report on safety of new designs. Forwards article on behavior of concrete elements subjected to tri-directional shear. Forwards PowerPoint presentation on good engineering practice again.	For Information and panel awareness. Technical information noted by the Panel.
13) 11/29/2012 Ma to Panel (11:25 am)	Forwards revised SOC	Subsumed by final SOC.
14) 11/29/2012 Ma to Hackett (4:43 pm)	Concurs on SOC	SOC is finalized and forms the basis for the Panel's review.
15) 12/26/12 Ma to Panel	Provides list of materials he recommends be provided to Panel contractors. Forwards email previously sent to [redacted] and B. Tomas which in turn forwards his Memorandum of Understanding on the Resolution of Ductility Issue of SC Wall from June 2010 and preliminary staff feedback on SB part 1 report.	Contractor tasking was decided by the Panel.
16) 01/17/2013 Ma to Panel	Forwards book chapter on tri-directional shear. Reiterates argument previously stated in DPO paper.	Subsumed by final SOC. Technical information noted by the Panel.

17) 01/22/2013 Ma to Panel	Forwards articles on minimum code requirements and fostering a culture of safety. Forwards PowerPoint presentation on good engineering practice. States that bad engineering practice was employed in AP1000 SB design. Reiterates arguments regarding tie-bar spacing and ductility ratio.	Subsumed by final SOC. Technical information noted by the Panel.
18) 02/11/2013 Ma to Panel	Forwards technical paper (Mullapudi) on aircraft impact and states that Case 4 in that paper supports his argument that the AP1000 design is not adequate.	Technical information noted by the Panel. Mullapudi paper is referenced in the DPO Panel Report.
19) 02/15/2013 Ma to Pedersen	Discusses communications with Panel and Office Director.	Beyond scope of the Panel's Charter – OE/NRO purview.
20) 03/18/2013 Ma to Panel	Forwards comments on Panel's contracts	No action – selection of the contractors is decided by the Panel
21) 03/28/2013 Ma to Panel	Forwards Canadian Nuclear Safety Commission (CNSC) questions on SB. Forwards his analysis of and answers to these questions.	Technical information noted by the Panel.
22) 04/01/2013 Ma to Wunder	Asks what he should forward to AP1000 mailbox.	Resolved in subsequent email communication between Wunder and Ma
23) 04/11/2013 Ma to Panel	Forwards analyses of inherent torsion, accidental torsion and anchor bolt stiffness.	Technical information noted by the Panel. Topics noted are addressed in Report Section 4.3.
24) 04/16/2013 Ma to Hackett	Accepts invitation to meet with Dr. Rashid. Dr. Ma had also previously met with Mr. Wyllie at the Panel's invitation.	Action completed.
25) 04/23/2013 Ma to Panel	Forwards a PowerPoint presentation on SB aircraft impact	Technical information noted by the Panel.
26) 04/26/2013 Ma to Hackett	Responds to question from Hackett and provides paper by Wight and Sozen.	Technical information noted by the Panel. Wight and Sozen paper is referenced in the DPO Panel Report.

27) 05/03/2013 Ma to [redacted]	Forwards source documents in response to question.	Action completed.
28) 08/29/2013 Ma to Panel	Forwards design guide for walls and reiterates DPO concern on GDC-2.	Technical information noted by the Panel. GDC-2 concern is included in the SOC.
29) 11/04/2013 Ma to Hackett	Summarizes DPO. Forwards individual's paper of 06/29/12 again (see email 3). Forwards paper by Vecchio again (see email 3). Forwards paper by Hsu again (see email 3). Forwards page of article on wall design; email Ma to Tegler dated 06/11/11 discussing two failure modes due to aircraft impact; email Ma to Tegler dated 05/24/11 on aircraft impact; figure regarding ductile energy absorption; chain of emails ending in Chuang to Ma dated 01/03/12 on ACI Code punching shear (this email chain includes conclusions by Chuang); calculation on punching shear; article by Rai on elevated tanks; individual's 10/22/10 rebuttal of staff response to DPV; NEHRP recommended seismic provisions; table and graph regarding material properties; article on ethics.	Technical information noted by the Panel.
30) 12/28/2013 Hackett to Tracy	Officially transmits concerns from Wyllie letter of 12/13/2013 to NRO	Noted in the Panel Report
31) 01/17/2014 Ma to Panel	Summarizes and re-emphasizes DPO concerns	Issues raised are included in the SOC and are addressed by the Panel
32) 01/21/2014 Hackett to Ma	Acknowledges receipt of Ma email of 01/17/2014	For information and Panel awareness

33) 01/29/2014 Ma to Panel	Summarizes and re-emphasizes DPO concerns	Issues raised are included in the SOC and are addressed by the Panel
34) 02/04/2014 Ma to Panel	Compared to other new NPP designs such as the latest German Konvoi, EPR, ATMEA1 and EC6 which are approximately 6 feet thick, it is contended that the AP1000 has insufficient thickness.	The DPO Report Section 1.0 "Introduction" lists the specific tasks assigned to the DPO Panel by the NRO Office Director. Task (d) is to "Maintain the scope of the review within the original DPO and SOC." One of the SOC issues the Panel has addressed is whether or not the cylindrical shell of the AP1000 SB meets the AIA requirements of 10 CFR 50.150. Based on three independent AIAs conducted by WEC, RES and the Panel's expert, the Panel concludes that the AP1000 SB cylindrical shell meets the AIA requirements. As such, it is neither necessary, nor is it within the Panel's scope to address why the cylindrical shell of the AP1000 SC panel may not be as thick as other NPP SB RC designs.
35) 02/06/2014 Ma to Panel	Forwards article about brittle fracture of marine vessel and a reference to a staff evaluation of the SB wall for a tornado missile	Technical information noted by the Panel

Appendix E- Crosswalk of DPO issues with Important Email Communications

1. The shield building wall design doesn't meet ACI building codes:

    a. Staff claimed that the ACI Code was only suggested, not required, as part of the SRP and RGs

    This is a subject of emails 1, 2, 6, 17, and 28. The issue is addressed in Executive Summary item 3(b) and Panel report sections 4.3.2(b)I and 4.3.2(b)ii

2. The staff approved the SC structure for the SB wall that failed in a brittle manner during testing or did not exhibit adequate ductility:

    a. Staff misinterpreted the ACI Code as having no ductility requirement. This is a subject of emails 3, 6, 10, and 17. This is addressed in Executive Summary item 3(c) and Panel report section 4.3.2(b)iii.

    b. The staff didn't provide technical justification for acceptance of a brittle SC element; furthermore, this acceptance is contrary to the acceptance criterion agreed upon with WEC. This is a subject of emails 3, 6, 10, 17, and 30. This is addressed in Executive Summary item 3(d) and Panel report sections 4.2.3.1 and 4.3.2(b)iv.

    c. The Brittle module is less ductile, weaker, and can't absorb/dissipate energy as effectively; bad for cyclic and impact loadings. This is a subject of emails 3, 4, 6, 10, 17, 30, and 33. This is addressed in Panel Report section 4.2.3.1.

    d. The shattering effect of brittle concrete hasn't been investigated. This is a subject of emails 3, 6, 10, 17, and 33. This is addressed in Panel report section 4.2.3.1.

3. The staff's conclusion of no aircraft penetration/perforation is illogical:

    a. WEC should demonstrate that the shield building possesses HCLPF against the fall of the water tank through the roof; however, what WEC has provided in this area is inadequate. This is a subject of email 18. This is addressed in Executive Summary item 2 and in Panel report section 4.2.4.1 and 4.2.4.2.

    b. It contradicts the analysis done for the ESBWR and ABWR. This is a subject of emails 3, 4, and 30. This is addressed in Panel report sections 4.2.3.1, 4.2.3.2, and 4.2.3.3.

    c. The material is brittle, and affords little punching shear strength and deformability, and it is susceptible to shattering. This is a subject of emails 3, 6, and 18. This is addressed in Panel report sections 4.2.3.3, and 4.2.3.4.

    d. Wall thickness should be greater than 4 feet. This is a subject of emails 3, 6, 10, 18, 21, 30, and 33. This is addressed in Panel report Section 4.2.3.

4. The AP1000 shield building does not meet the NRC's seismic margin requirement:

    a. In-plane shear stress greater than ultimate in-plane shear strength for entire wall height for Review Level Earthquake (RLE). This is a subject of emails 3 and 28. This is addressed in Executive Summary item 1 and in Panel report section 4.1.

    b. Some analyses weren't performed when considering forces/stresses for earthquake loading: irregular shape at concrete boundary, accidental torsion effect, second-order P-Delta effect, interaction relationships. This is a subject of emails 3, 6, 23, and 30. This is addressed in Executive Summary item 3(a) and in panel report section 4.3.2(a).

    c. Wall thickness should be greater than 3 feet. This is a subject of emails 3, 6, 21, 30, and 33. This is addressed in Panel report sections 4.1.1, 4.1.4, 4.2.3, and 4.3.2.

5. WEC's use of nonlinear static pushover analysis in lieu of ACI Code was inappropriate for analyzing collapse/seismic behavior:

    a. Constitutive laws are not available for SC concrete elements; therefore, WEC had no basis for using the codes they used. This is a subject of emails 1, 2, and 6. This is discussed in Panel report section 4.2.3.5.

    b. Static analyses don't account for dynamic (seismic) events; therefore, the use of the static pushover analysis is not vaild. This is a subject of emails 3, 6, and 10. This is addressed in Executive Summary item 3(e) and in Panel report section 4.3.2(b)v.

    c. Building codes prohibit the use of a static pushover analysis because the building is irregular, essential, and over 40 feet high. This is a subject of emails 3 and 10. This is addressed in Executive Summary item 3(e) and in Panel report section 4.3.2(b)v.

6. The shield building wall is insufficiently strong and ductile to resist earthquakes or aircraft missiles:

    a. WEC has not provided a valid demonstration with respect to ductility and energy dissipation/absorption of the shield building. This is a subject of email 9. This is addressed in Executive summary item 3(d) and in Panel report sections 4.2.3.1 and 4.3.2(b)iv.

    b. A more complete analysis would have shown that in-plane shear forces could exceed 100 times $\sqrt{f_c{}'}$ This is a subject of email 6. This is addressed in Panel report section 4.1.2.4

    c. In-plane shear strength testing should have been performed on special equipment at University of Houston. This is a subject of email 4.

    d. The connection of the water tank to the shield building roof is insufficient. An aircraft impact could cause the tank to fall, break through containment, and possibly cause core damage:

        This is a subject of emails 3, 6, 12, and 28. This is addressed in Executive Summary item 2(a) and in Panel report section 4.2.4.2(a)

7. The AP1000 shield building has not been shown to satisfy GDC 2. This is a subject of emails 3 and 28. This is addressed in Executive Summary item 4 and in Panel report section 4.4.2.

Document 4: Supplemental Tasking Memo from Office Manager to DPO Panel

August 14, 2014

MEMORANDUM TO: Edwin M. Hackett, Chair
Differing Professional Opinion
Ad Hoc Review Panel

[redacted] Member
Differing Professional Opinion
Ad Hoc Review Panel

[redacted], Jr., Member
Differing Professional Opinion
Ad Hoc Review Panel

FROM: Glenn M. Tracy, Director ***/RA/***
Office of New Reactors

SUBJECT: AD HOC REVIEW PANEL – DIFFERING PROFESSIONAL OPINION ON THE AP1000 SHIELD BUILDING (DPO-2012-002)

By memorandum dated September 12, 2012, I appointed you as members of a Differing Professional Opinion (DPO) Ad Hoc Review Panel and tasked you to issue a DPO report, including conclusions and recommendations to me regarding disposition of the issues presented in the DPO.

On February 24, 2014, you provided me with a report of the Panel's views. The Panel's report contains several observations and recommendations regarding the design of the AP1000 shield building. Per Management Directive 10.159, "The NRC Differing Professional Opinions Program," I reviewed the Panel's report in order to make a decision regarding the merits of the issues raised by the submitter, and determined there was a need for clarification regarding several matters within the report.

I met with the Submitter on May 1, 2014, and we discussed my proposed approach to conduct detailed meetings among the staff and the DPO Panel to facilitate discussion on these matters within the DPO Panel's Report. I led such meetings on May 19, 2014 and June 5, 2014, with supporting agency executives, the DPO Panel, representatives from the Office of Enforcement, the Office of General Council, the Office of New Reactor's (NRO) Open Collaborative Work Environment (OCWE) group, and staff experts from NRO and Research. During the meetings, important information was discussed that requires further deliberation by the DPO Panel, as noted by the Panel. On June 19, 2014, I held a meeting with the same individuals to discuss the process forward and, later in the day, held a similar meeting with the Submitter and the NRO OCWE Champion.

Based on the activities subsequent to issuance of the Panel's report, I am tasking the DPO Panel to supplement its report to document its views on the information that was discussed during the May 19 and June 5 meetings; the July 24, 2014 memorandum documenting the staff response to the DPO 2012-002 Panel report; and any relevant additional information that will be presented by the Submitter in a near-term meeting with the Panel that will be scheduled prior to the Panel's final deliberations in this tasking. I request the Panel's supplemental report by September 30, 2014.

I sincerely appreciate your continued dedicated efforts to complete an independent and objective review of this complex DPO, and I look forward to receiving your final views and recommendations. Your efforts, as well as those of the staff and Submitter, continue to model the agency's values toward the attainment of a sound and independent decision in this important matter.

cc: Submitter
R. Pedersen, OE/DVPM

presented by the Submitter in a near-term meeting with the Panel that will be scheduled prior to the Panel's final deliberations in this tasking. I request the Panel's supplemental report by September 30, 2014.

I sincerely appreciate your continued dedicated efforts to complete an independent and objective review of this complex DPO, and I look forward to receiving your final views and recommendations. Your efforts, as well as those of the staff and Submitter, continue to model the agency's values toward the attainment of a sound and independent decision in this important matter.

cc: Submitter
R. Pedersen, OE/DVPM

ADAMS: ML14223B223 *via e-mail

OFFICE	NRO/DNRL/LB1	NRO/OCWE	OE/DVPM:	OGC	NRO DD	NRO
NAME	GWunder	JSteckel	RPedersen*	SKirkwood*	GHolahan	GTracy
DATE	08/14/2014	08/14/2014	08/14/2014	08/14/2014	08/14/2014	08/14/2014

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Document 5: Supplemental Panel Report

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, DC 20555 - 0001

October 30, 2014

MEMORANDUM TO: Glenn Tracy, Director
Office of New Reactors

FROM: E. M. Hackett, DPO Panel Chair /RA/
[redacted], DPO Panel Member /RA/
[redacted], DPO Panel Member /RA/

SUBJECT: DIFFERING PROFESSIONAL OPINION PANEL SUPPLEMENTAL REPORT ON DPO-2012-002 – STRUCTURAL INTEGRITY CONCERNS WITH THE AP1000 SHIELD BUILDING

In a memorandum dated September 20, 2012, you appointed us as members of a Differing Professional Opinion (DPO) Ad Hoc Review Panel (DPO Panel) to review a DPO regarding structural integrity concerns with the AP1000 shield building. The DPO Panel reviewed the DPO in accordance with the guidance in Management Directive (MD) 10.159 and provided a report with our findings to you on February 24, 2014.

Per MD 10.159, you reviewed the Panel's report in order to make a decision regarding the merits of the issues raised by the submitter, and you determined there was a need for clarification regarding several matters within the report.

With the support of the Office of Enforcement (OE), you convened two meetings among the staff and the DPO Panel to facilitate detailed discussion on these matters within the DPO Panel's Report. These meetings took place on May 19 and June 5, 2014, with supporting agency executives, the DPO Panel, representatives from OE, the Office of General Counsel (OGC), the NRO Open Collaborative Work Environment (OCWE) representatives, and staff experts from NRO and the Office of Research (RES). During the meetings, important information was discussed that required further deliberation by the DPO Panel, as noted by the Panel. On June 19, 2014, you held a meeting with the same individuals to discuss the process forward. You directed your staff to develop a report summarizing the deliberations for the Panel's review. This report was completed on July 24, 2014 and was forwarded to us. The DPO submitter was kept informed of the process throughout.

-2-

Based on the above activities, you tasked us to supplement our report to document our views on the supplementary information that was discussed during the May 19 and June 5 meetings; the July 24, 2014, staff response to the DPO 2012-002 Panel report; and any relevant additional information presented by the Submitter in a meeting with the Panel on September 08, 2014.

We have completed our assessment of the supplementary information presented to us and our views and findings are documented in the Enclosures. With some augmentation, the Enclosures mirror those from the NRO staff's report of July 24, 2014 and retain the staff's assessment in addition to our supplementary evaluation. In particular, Enclosure 2 contains the item-by-item description of the issues identified in the DPO Panel Report for follow-up, the staff's assessments and conclusions, and our supplementary evaluation.

In summary, the Panel finds that the staff followed-up on the Panel's recommendations and provided supplementary information for our consideration. Overall, the Panel finds that the staff's resolution is acceptable with a view that certain issues (Items 2, 4 and 14 in Enclosure 2) may merit further consideration.

The Panel appreciates the additional efforts in this matter put forth by NRO, the DPO Submitter and all other parties involved in order for the Panel to conclude its' review of the DPO.

Enclosures:

1. High Level Summary of Issues Identified for Follow-up in the DPO Panel Report and the Panel's Supplementary Evaluation

2. Item-by-Item Staff Assessment of follow-up activities identified in the DPO Panel Report and the DPO Panel's Supplementary Evaluation

3. NRO/DE Evaluation of the Effects of Cold Temperature on the AP1000 AIA and the Supplementary Assessment by DPO Panel Member [redacted] (Item 3, Enclosure 2)

4. NRO/DE Evaluation of AIA Benchmarking and the DPO Panel's Supplementary Assessment (Item 15, Enclosure 2)

5. NRO/DE Evaluation of the Submitter's Response (May 1, 2014) to the DPO Panel Report and the DPO Panel's Supplementary Evaluation

6. NRO/DE Response to the Submitters May 28, 2014 Email Comments on Curvature Effects and the DPO Panel's Supplementary Evaluation

7. DPO Panel's Assessment of the Submitter's Email Comments of March 27, 2014

8. DPO Panel's High Level Summary of the September 8, 2014 Meeting with the Submitter

9. DPO Panel's Assessment of the Staff's AIA Inspection of the PCS Tank and Containment (Item 4, Enclosure 2)

10. DPO Panel's Assessment of Supplementary Information Regarding the Staff's Seismic Evaluation in FSER Section 3.7 (Item 14, Enclosure 2)

cc: J. Ma, NRO
R. Zimmerman, DEDO
P. Holahan, OE
R. Pedersen, OE
M. Sewell, OE
D. Solorio, OE
G. Holahan, NRO
G. Wunder, NRO
J. Williams, NRO
[redacted] NRO
J. Steckel, NRO
J. Tappert, NRO
[redacted]
A. Bellinger, ACRS
S. Meador, ACRS
S. Kirkwood, OGC
B. Stapleton, NSIR
D. Dorman, NRR
J. Pires, RES

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D. Dorman, NRR
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Accession No: **ML14307A870** **Publicly Available** **N** **Sensitive** **Y**
Viewing Rights: DPO Panel and ☒ See Restricted distribution

OFFICE	DPO	DPO	DPO	
NAME	[redacted]	[redacted]	EMHackett	
DATE	10/30/14	10/30/14	10/30/14	

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Enclosure 1 - High Level Summary of Issues Identified for Follow-up in the DPO Panel Report and the Panel's Supplementary Evaluation

The structure of this, and the subsequent Enclosures, mirrors the structure of the staff's report dated July 24, 2014. That is, we considered each topic discussed by the staff in order, the staff response, and our assessment of the adequacy of the response relative to issues we raised in our original Panel Report dated February 24, 2014. Enclosure 2 provides an item-by-item detailed staff response to these and other recommendations in the DPO Panel Report, along with the Panel's supplementary evaluation.

<u>Topic: Seismic Margin</u>

Concern: Panel expressed concern that high confidence of low probability of failure (HCLPF) values in the Design Control Document (DCD) did not incorporate bounding operating temperature (-40˚F in the case of AP1000).

Staff Response:

- There is no requirement for Westinghouse Electric Corporation (WEC) to include temperature effects in the calculation of DCD HCLPF values. HCLPF values are used in beyond-design-basis seismic-margin calculations and make use of seismic and gravity demands only.
- Accordingly, the DCD HCLPF values for the Shield Building (in Table 19.55) remain the design basis and these values continue to be conservative (>0.5 g PGA, the AP1000 seismic margin target for peak ground acceleration) and acceptable.

DPO Panel's Supplementary Evaluation:

The Panel considers the follow-up to be acceptable and the issue to be resolved. Based on the review of the supporting information referenced by the staff (ASME/ANS Standard RA-Sa–2009), the Panel agrees with the staff and concludes that WEC need not re-evaluate the HCLPF values consistent with reanalysis results reported in Appendix L of the Shield Building Report that include thermal effects.

Concern: Panel expressed concern with a few discrepancies between the tabulated HCLPF values in the DCD and in the Shield Building report.

Staff's Response:

- For most cases, the DCD values are conservative estimates as compared to those in the Shield Building report. They can be conservative by as much as [ ] [a,c] The DCD value is more conservative because [ ] [a,c]

- When the DCD values are more than those cited in the Shield Building report, the differences cited are less than [ ] a,c HCLPF differences are expected because HCLPF calculations are merely estimates that depend on input assumptions. The differences are small and have minimal safety significance because the values reported in both documents are greater than 0.5 g (the safety goal).

- Based on the low safety significance and the finding that the HCLPF values in both documents are conservative (i.e., greater than 0.5 g peak ground acceleration (PGA)), staff considers this issue resolved.

DPO Panel's Supplementary Evaluation:

The Panel considers that this issue is not significant relative to the DPO concerns. We raised the issue relative to appropriate QA follow-up and defer to the staff for implementing the appropriate NRC QA requirements. The Panel notes that WEC acknowledged the discrepancy to the Panel and committed to enter this item in their corrective action program.

Topic: Detailed Design

Concern: Panel expressed concern with eight (8) areas involving detailed design of the AP1000 Shield Building: (1) Air intake region, (2) steel concrete composite (SC) wall connections, (3) the connection between the Shield Building wall and the Auxiliary Building roof, (4) passive containment cooling system (PCS) tank design temperature, (5) PCS tank vertical seismic, (6) PCS tank shear reinforcement, (7) PCS tank construction voids, and (8) stability of the shield building compression ring.

Staff's Response:

Staff performed a follow-up review of each area and found that the design continues to satisfy the regulations.

- Items (6) and (8) are the two significant items discussed and resolved with the panel consultant, Mr. Wyllie, on January 24, 2014. Staff audited the PCS tank calculations and adequately addressed the thermal, hydrodynamic, and vertical seismic loads. This review is documented in the U.S. Nuclear Regulatory Commission (NRC) staff's Final Safety Evaluation Report (FSER) (Agency-wide Documents Access and Management System (ADAMS) Accession No. ML112061231).Quality of concrete (the voids) and the constructability of the air intake region (and others) are addressed through the mockup testing program.
- The Auxiliary-Building-to-Shield Building connection remains acceptable based on the staff's independent assessment and the combined license (COL) requirement to ensure that detailed design is performed in accordance with the licensing basis methods, codes and standards.

- Staff has reasonable assurance that the Shield Building design is safe based on: (a) the use of cited codes and standards for the detailed design, (b) the use of structural models with adequate refinement leading to reasonable determination of demands, (c) acceptable demand-to-capacity values, and (d) a commitment to perform pre-construction mockups to provide assurance of concrete placement quality.

DPO Panel's Supplementary Evaluation: See Enclosure 2, Items 4-13

Topic: Aircraft Impact Assessment (AIA)

Concern: Panel expressed concern that it was not able to determine whether WEC had met the intent of subsection 2.4.1(4) of Nuclear Energy Institute (NEI) 07-13, "Methodology for Performing Aircraft Impact Assessments for New Plant Designs," with regards to benchmarking of computer models for AIA.

Staff's Response:

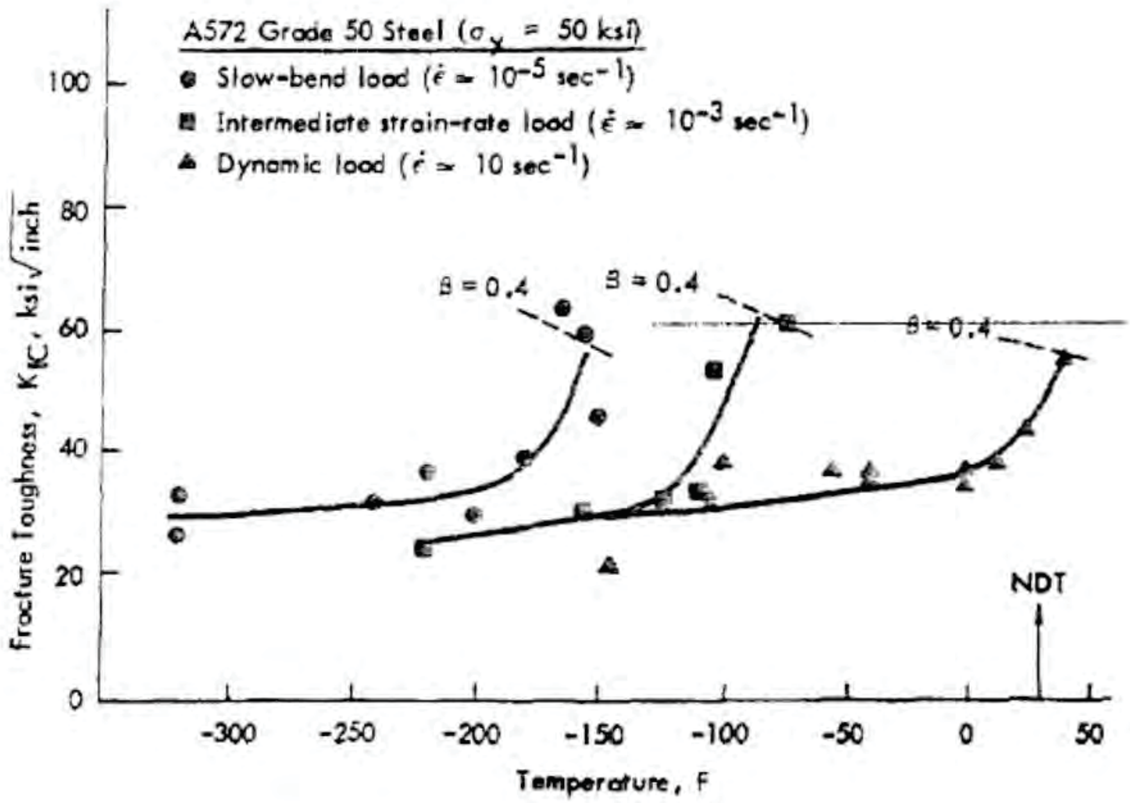
- Staff performed an inspection of the WEC AIA in 2010 and specifically focused on analysis benchmarking. An independent consultant from Sandia National Laboratory (SNL) supported the staff's review of this area. The inspection report (ADAMS Accession No. ML102980583) states that the NRC inspection team reviewed the benchmarking process and its technical justification and verified them to be accurate and complete. Staff also performed followup AIA inspections in May, June, and August of 2011, which are documented in an October 2011 inspection report (ADAMS Accession No. ML112650748).
- Staff also notes that other independent analysis models, performed by NRC's Office of Nuclear Regulatory Research (NRC/RES) and Dr. Rashid (the contractor hired by the panel), have undergone their own independent benchmarking and have arrived at the same overall conclusion (no perforation of the Shield Building). The staff views these findings by the independent analysts as another element of reasonable assurance in the adequacy of the WEC AIA.
- Based on the NRC inspection findings, the staff continues to believe that the criteria of subsection 2.4.1(4) of NEI 07-13 have been satisfied and that the AP1000 Shield Building AIA was performed in accordance with regulatory guidance and therefore satisfies the regulations.

DPO Panel's Supplementary Evaluation: See Enclosure 2, Item 15 and Enclosure 4

Concern: Panel member [redacted] expressed concern that the ability of the Shield Building wall to resist aircraft impact under cold weather has not been demonstrated and is doubtful.

Staff's Response:

- There is no cliff-edge effect at 30°F. Consideration of the best-estimate cold temperature in the United States is that it will have little to no effect on the Shield Building AIA analysis results, because the nil-ductility transition (NDT) of A572 steel (the steel used in the Shield Building) is much less than 30°F for aircraft impact strain rates (see Fig. 1 – see page 8).
- Based on its regulatory and technical review of the concern, the staff agrees with the DPO report position that the AP1000 Shield Building is competent to stop a large commercial aircraft without perforation of the wall or a threat to the integrity of the containment.
- Further, staff agrees with the DPO panel majority position (Hackett/[redacted]) that -40°F is a lower-bound temperature and results in lower-bound material properties. The use of such lower-bound properties is not in conformance with the AIA rule, the Statement of Consideration (SOC) of the AIA rule and implementing guidance, industry codes and standards for impact problems, and civil engineering practice for both design-basis and beyond-design-basis events. Enclosure 3 to this memo provides additional information on this issue.
- Further, recognizing the scope of uncertainty in AIA and the defense-in-depth in the regulations, further precision in one parameter (ambient temperature) would have little to no effect on increasing the overall accuracy of the assessment.
- Based on the above, staff finds the DPO panel's concerns relating to the AP1000 Shield Building AIA to be fully addressed.



Effect of temperature and loading rate on fracture toughness of A572 Grade 50 steel (σ_{ys} = 50 ksi)

Figure 1. Effect of Temperature on A572 Steel (from DPO Panel Report)

DPO Panel's Supplementary Evaluation: See Enclosure 2, Item 3 and Enclosure 3

Enclosure 2. Item-by-item Staff Assessment of Follow-up Activities Identified in the DPO Panel Report and the DPO Panel's Supplementary Evaluation

The staff's review of the Differing Professional Opinion (DPO) Panel Report identified several key issues for which a follow-up was recommended by the DPO Panel, including seismic margin, detailed Shield Building (SB) design, and aircraft impact assessment under cold temperature conditions. For this enclosure, we have copied the assessment from the staff's report of July 24, 2014 and added a column to provide the Panel's Supplemental Evaluation. A total of 19 items are specifically addressed.

Item No.	DPO Report Section	Item	Staff Assessment	Conclusion	***Panel's Supplementary Evaluation***
1	6.1	Westinghouse Electric Company (WEC) did not re-evaluate the high confidence of low probability of failure (HCLPF) values in a way consistent with the Shield Building (SB) reanalysis results that are reported in Appendix L of the Shield Building Report (DPO Panel Report, Section 4.1.3).	Seismic margin assessment is performed to ensure that structures, systems, and components (SSCs) important for safe shutdown have margin beyond the design basis. The assessment of seismic margin, unlike design-basis analysis, is performed using best-estimate material properties and allows material aging and ductility to be credited. While seismic margins analysis (SMA) considers normal loads such as live and dead loads, there is typically no consideration of extreme ambient temperature effects (-40˚F and 115˚F in the case of AP1000). Staff notes that ASME/American Nuclear	**Staff considers this issue resolved.** Staff has performed a follow-up review of this issue and concludes that WEC is not required to include temperature effects in the calculation of Design Control Document (DCD) HCLPF values. Accordingly, the DCD HCLPF values for the SB (Table 19.55) remain the design basis and these values continue to be conservative (> 0.5 g, the AP1000 seismic margin target for peak ground acceleration (PGA)) and acceptable. This position is consistent with the DPO panel's conclusion that the AP1000 SB does meet the NRC	***The Panel considers the follow-up to be acceptable and the issue to be resolved.*** *Based on the review of the supporting information referenced by the staff (ASME/ANS Standard RA-Sa–2009), the Panel agrees with the staff and concludes that WEC need not re-evaluate the HCLPF values consistent with reanalysis results reported in Appendix L of the Shield Building Report that include thermal effects.*

Item No.	DPO Report Section	Item	Staff Assessment	Conclusion	*Panel's Supplementary Evaluation*
				Society (ANS) RA-S-2009, endorsed in staff guidance, recommends that low winter temperatures not be considered in seismic margin analysis because thermal stresses and embrittlement are usually insignificant or covered by design codes and standards for plant design. This is the case for the AP1000 design, in which thermal stresses were considered as part of the load combinations for the design-basis analyses, including seismic analyses. Appendix L reports the results of the design-basis load combination of normal operating thermal and seismic loads as required in American Concrete Institute (ACI) 349-01. Although the design margin decreased because the thermal load case was	seismic margin requirements (DPO report, page 6). No further action is recommended.

Item No.	DPO Report Section	Item	Staff Assessment	Conclusion	*Panel's Supplementary Evaluation*
			included, this change in design margin has no impact on the HCLPF values described in the AP1000 DCD or the SB report (Chapter 11) because HCLPF values are developed to assess seismic margin and are independent of normal operating temperature conditions as discussed above. Staff agrees with the WEC position that the DCD HCLPF values will not be changed as a result of the thermal reanalysis in Appendix L to the SB report (DPO report, page 24).		
2	6.2	The HCLPF values reported in Table 19.55-1 of the DCD are at variance with the ones reported in Chapter 11 (pages 16 through 23) of the Shield Building design report (DPO Panel Report Section 4.1.2.2).	For the AP1000 DCD review of seismic margin as required under the Staff Requirements Memorandum (SRM) for SECY 93-87, staff relied on the DCD HCLPF values for the Shield Building reported in DCD	**Staff considers this issue resolved**. Staff has performed a follow-up review of this issue and concludes that while there is a variation in the reported HCLPF values, the variation has minimal safety significance. The	***The Panel considers that this issue is not significant relative to the DPO concerns. We raised the issue relative to appropriate QA follow-up and defers to the staff for***

Item No.	DPO Report Section	Item	Staff Assessment	Conclusion	*Panel's Supplementary Evaluation*
			Table 19.55. The staff review focused on the WEC methodology for developing HCLPF values and ensuring that HCLPF values were greater than 0.5 g (1.67 × 0.3 g) for SSCs listed in DCD Table 19.55. An audit was performed and focused mainly on components with lower HCLPF values, such as the pressurizer upper support weld (HCLPF = 0.5 g). Seismic margin experts Goutam Bagchi (NRC) and Abhinav Gupta (North Carolina State University) supported the staff's review. The staff did not rely on the SB report's HCLPF values for the design-basis safety evaluation of the Shield Building. There is no mention of Shield Building HCLPF values in either DCD Chapter 3 or the corresponding staff FSER.	worst case variation of [ ] [a,c] indicates a conservative DCD HCLPF value. Regardless of the variation, all values in question satisfy the seismic margin requirements for the design (>0.5 g). As such, the staff continues to believe that both the seismic margin calculations in the DCD and those in the SB report provide reasonable assurance that the SB possesses adequate seismic margin. Based on the low safety significance and conservative HCLPF values for the SB design, staff considers this issue resolved. No further action is recommended.	***implementing the appropriate NRC QA requirements. The Panel notes that WEC acknowledged the discrepancy to the Panel and committed to enter this item into their corrective action program.***

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				Staff review finds that the SB report (in Chapter 11) indicates [ ] a,c values (relative to the DCD) for the steel concrete composite (SC)-reinforced concrete (RC) connection and the air-inlet region. However, these differences are [ ] a,c respectively, and are judged to be minimal and within the uncertainty band of the calculations. Staff review finds that the SB report (in Chapter 11) indicates [ ] a,c values (relative to the DCD) for the tension ring and conical roof. While these differences are [ ] a,c respectively, they are conservative with respect to the DCD reported values relied on by staff in the FSER. The staff does not have the detailed calculations to assess the reason for the differences in values, but notes that variations in	

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			approach for calculating seismic margin are acceptable and will yield results with different degrees of conservatism. Staff also notes that for the conical roof, the DCD HCLPF calculations conservatively did not [ ]a,c while the SB report did. This is likely the reason for the [ ] a,c difference in HCLPF results. In addition, staff notes that both the DCD and SB report HCLPF values for the tension ring and conical roof [ ] a,c are significantly more than the required value of 0.5 g.		
3	6.3	Effect of cold temperatures on the Shield Building aircraft impact assessment (AIA): Panel member [redacted] recommends that the staff follow up on this issue (DPO Report Section 4.2.3.5.1(b)).	See Enclosure 3.	See Enclosure 3.	***Based on the review of the staff's supporting information in the report and the RES independent AIA analysis of the AP1000 shield***

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					building, Panel member [redacted] concludes that the Shield Building wall is expected to resist aircraft impact under cold weather in the United States (See Enclosure 3)
4	6.4	The Staff did not evaluate technical issue (c) in Safety Concern # 2 of the DPO Statement of Concerns (SOC) and did not inspect the WEC passive containment cooling system (PCS) tank and the containment assessment for the physical and shock effects of the aircraft impact (DPO Report Sections 4.2.4, 4.2.4.1, and 4.2.4.2).	As articulated in their January 19, 2011 letter, the Advisory Committee on Reactor Safeguards (ACRS) reviewed additional WEC AIA analyses performed to address concerns regarding the PCS tank impacts. The committee wrote that the WEC AIA satisfied the regulations (in 10 CFR 50.150, "Aircraft Impact Assessment"), but recommended that staff review the information presented by WEC to ascertain whether additional inspection was required.	**Staff considers this issue resolved.** Staff has performed a follow-up review of this issue and continues to believe that the AP1000 Shield Building AIA was performed in accordance with regulatory guidance and therefore satisfies the regulations. The AP1000 Shield Building AIA, including the PCS tank and shield plate analyses, were inspected by staff on several occasions and found to be performed in accordance with the regulatory guidance. Inspection reports documenting the various staff inspections of	***The Panel understands that the principal finding on this issue from WEC and the staff is that the PCS tank is not penetrated by the aircraft impact. In subsequent analyses conducted by WEC in response to ACRS questions, WEC maintained that even if structural failure of the PCS tank were to occur, the debris generated would be insignificant and would not cause failure of either the radiation shield plate or the containment.***

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				Subsequent to the January 2011 ACRS letter, the staff performed several additional AIA inspections (May 2011, June 2011, and August 2011). These inspections focused on a number of AIA-related areas, included the WEC analysis of the PCS tank and shield plate, and are documented in an October 2011 inspection report (ADAMS Accession No. ML112650748). Inspections included the particular impact simulation involving the PCS tank. While staff identified technical issues with assumptions in the LS-DYNA code (about time-step and mesh size) and documented those issues in the corresponding inspection reports, the staff found that the PCS tank AIA continued to be acceptable.	the AP1000 AIA are in ADAMS. No further action is recommended.	***The ACRS was satisfied with the WEC response on this issue and the staff stated that they conducted further inspections of the WEC assessments to provide added assurance. Following further deliberation on this issue, the Panel finds no reason to doubt the assessments made by the staff, WEC and ACRS on this issue.*** ***However, we do note that we were not able to perform our own independent assessment of this issue due to the lack of availability of certain portions of the documentation (Safeguards information maintained at the applicant's site) and***

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			The staff has no comments on the PCS tank failure mode postulated by the DPO panel (on page 45 of the panel report), but notes that the simulation performed by WEC was detailed enough to predict the failure mode of concern if it was plausible. Based on the staff's observations during several AIA inspections, the models showed that the mode of failure of concern associated with the PCS tank is not credible. Staff notes that the details of scenarios and results of the WEC AIA are Safeguards Information and should be discussed with appropriate protocols.		***general resource limitations. Therefore, the Panel relied on: (1) interviews with WEC and NRC staff, and (2) appropriate documentation (inspection reports and the ACRS letter on the subject). As noted in Enclosure 9, the inspection reports that the Panel reviewed did not contain documentation at a specific enough level to confirm details of the staff's inspection of the PCS tank impact assessment and any postulated debris impact on the containment.*** **(See Enclosure 9 for further details of the Panel's evaluation.)**
5	7.1	<u>Design of the SC Shield Wall Air-Intake Region</u>	The panel states (on page 83 of the DPO report) that the constructability of the	**Staff considers this issue resolved.** Staff has performed a follow-up review of this issue and	***The staff committed to resolve this issue through a framework that includes***

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		The DPO panel recommends that the staff follow up on this issue to ensure that the horizontal component of combined design-basis force at both transition bends is adequately resisted by the tie bars and that the air-intake region is properly designed and detailed and is constructible.	tie-bar detail for the approved certified design is not yet established. The issue of constructability is addressed through the pre-construction mockup program. The issue of the specific details of the tie bar is addressed, as are many other complex details in the SB, through a framework that has been shown to be effective in identifying, correcting, and (on many occasions) avoiding construction issues. This framework includes ongoing constructability reviews by the licensees and targeted inspections by the inspectors (which are enhanced and informed by insights from the licensees' constructability reviews and ultimately by the DCD requirement to develop and construct the final design in accordance with the codes of record).	agrees with the panel conclusion that the SB design continues to meet the requirements of General Design Criterion (GDC) 2, "Design Bases for Protection against Natural Phenomena" (DPO report, page 9). The staff reviewed the concern and concludes that the topic has been adequately addressed during the licensing review and that there is reasonable assurance that the constructability will be handled in the current construction inspection framework. Staff has reasonable assurance that the SB design of the air-inlet region is safe based on: (a) the use of cited codes and standards for the detailed design of the tie bars, (b) the use of structural models with detailed air-inlet regions leading to reasonable	***ongoing constructability reviews by the licensees and focused NRC inspections. The staff plans to confirm, via the inspections, that the design is ultimately constructed to the DCD requirements.*** ***On the basis of the staff's commitments, the Panel considers the follow-up to be acceptable and the issue to be resolved.***

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				The staff evaluated the specific issue raised about the tie bars in the air-inlet region. The staff notes that the detailed design of tie bars in SB critical sections will be performed in accordance with cited codes and standards. The design demands are developed from the SB roof model as described in Appendix 3G, "Nuclear Island Seismic Analyses," to the DCD. The one-quadrant model of the Shield Building roof is shown in Figure 3G.2-16, "Quadrant Model of the Shield Building Roof," in the DCD. The model is constructed with solid and shell elements and contains structures from the exposed shield wall through the top of the Shield Building roof. The quadrant model is used for the equivalent static analysis of the Shield	determination of demands, (c) acceptable demand-to-capacity values, and (d) a commitment to perform pre-construction mockups to provide assurance of concrete placement. No further action is recommended, beyond the currently planned SB and mockup inspections.

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			Building roof. The results from this more detailed analysis are used in the design of the Shield Building roof and radial roof beams, tension ring, air-inlet structure, and PCS tank. The model is sufficiently detailed to adequately describe the load path in the air-inlet region. In Appendix 3H, "Auxiliary and Shield Building Critical Sections," to the DCD, Table 3H.5-9 (Sheet 2c of 3), "Shield Building Roof Reinforcement Summary (Air Inlet)," indicates a demand-to-capacity ratio of 0.63 for seismic load combinations at the air-inlet region. While thermal demands are not included in this ratio, staff notes that thermal demands are small in this region of the SB because natural air flows on the inside face of the SB at		

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				that elevation. Also, the [[redacted]] a,c Regarding the DPO panel concern that tie bars and studs may be omitted in local areas due to design features and other other obstructions, staff notes that this approach is acceptable on the basis that the impacted areas are small (local) and that the design will continue to meet code. This approach is consistent with other areas of the design (e.g., CIS modules). Regarding the DPO panel concern that the two percent is sufficient and reasonable to account for uncertainty in in the analysis results (DPO report page 84), staff notes that the two percent	

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			allowance is provided to the designer and constructer to address small variations in materials and analysis tools as construction proceeds. Such allowances are consistent with standard practice. Staff notes that the licensee(s) will be performing the detailed design (verifying adequacy of the tie-bar design) in accordance with cited codes and standards (e.g., American National Standards Institute (ANSI)/American Institute of Steel Construction (AISC) N690 and ACI 349-01). Also, pre-construction mockups of the air-inlet region will be used to provide assurance in the method of concrete placement.		
6	7.2	<u>SC Wall Connections</u>	Staff review finds that the existing connection design	**Staff considers this issue resolved.** Staff has performed a follow-up	***The Panel considers the follow-up to be***

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		The DPO panel noted that while Mr. Wyllie believes the SC-RC connections are adequate and have adequate capacity, he recommended potential enhancements to the WEC design details (DPO report, page 84). Mr. Wyllie also states that he was unsure how the compression in the steel plates will be transferred by the shear studs and steel ties over many feet (DPO report, page 84).	continues to satisfy applicable code requirements with margin and therefore constitutes a safe design. While Mr. Wyllie's suggestion may be an acceptable design alternative, there is no regulatory basis to require a change to the AP1000 DCD. Regarding Mr. Wyllie's concern with the transfer of compression in the steel plates, staff notes that the SB report, Section 3.2.2.11, states that the [ ] a,c The mechanism for transfer forces from the steel face plates to the concrete is through the use of shear studs, which is a common mechanism in concrete construction. The staff reviewed Mr. Wyllie's	review of this issue and agrees with the panel's conclusion that the SB design continues to meet the requirements of GDC 2. Further, staff agrees with Mr. Wyllie's conclusion that the current SC-RC connection's design and detailing are adequate and have adequate capacity. It is inappropriate for the staff to direct the applicant to change (backfit) a design that is already acceptable. No further action is recommended.	***acceptable and the issue to be resolved. The following provides the Panel's basis for this resolution:*** ***As noted by the staff, transfer of compression forces from the SC panel steel plates to the concrete is accomplished by shear transfer [ ] a,c*** ***At the connection, the concrete compressive forces bear directly on the [ ] a,c However, in the event that there is not solid concrete [ ] a,c***

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				recommendation to add an additional nut to the tie-rod connection. Based on the observation that the SC/RC design transfers compression from the steel plates to the concrete through shear studs located above the connection zone, the additional nut suggested by Mr. Wyllie may be helpful but does not appear necessary.	***[ ] [a,c] may not be completely transferred by direct compression into the concrete below. Instead, some of these forces will be transferred by an increase in the compressive stresses in the concrete between [ ] [a,c] In the that there is not solid concrete [ ] [a,c] the Panel believes that the***

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					load-path described above can be safely mobilized and will satisfy the allowable stresses. The Panel agrees with Mr. Wyllie's assessment that the connection has adequate capacity to resist the design loads
7	7.3	<u>Auxiliary Building RC Roof Connection to the SC Wall</u> The DPO panel noted that while Mr. Wyllie believes the auxiliary building roof connection is acceptable, a simpler detail could be implemented (DPO report, page 85). Mr. Wyllie also recommended that staff review the shear lug detail shown in SB report Figure 4.2-6,"Shear Lug Detail", because his simplified calculations indicate flexural stresses in	Staff review finds that the existing auxiliary building connection design continues to satisfy applicable code requirements with margin and therefore constitutes a safe design. While Mr. Wyllie's suggestion may be an acceptable design alternative, there is no regulatory basis to require a change to the AP1000 DCD. Regarding the shear lug detail and concern for high stresses in the SC plate, staff review finds that the analysis	**Staff considers this issue resolved.** Staff has performed a follow-up review of this issue and agrees with the panel's conclusion that the SB design continues to meet the requirements of GDC 2. Further, staff followed up on Mr. Wyllie's concern for high stress in the SC wall and continues to have adequate assurance in the current design of the connection. No further action is recommended.	***The Panel considers the follow-up to be acceptable and the issue to be resolved.*** ***In Mr. Wyllie's calculation he assumed the concrete bearing stresses were distributed uniformly over the [ ] a,c surface of the shear lug and calculated a bending moment at the centerline of the [ ] a,c In the staff's calculation the same bending moment***

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		excess of the plate yield stress (as stated on page 85 of the DPO report).	described in Section 4.2 of the SB report constitutes a level of information detail needed to make a safety determination of the Shield Building but it is not at a level of detail required to assess the adequacy of the detailed design (local plate stresses). The actual detailed design of the connection will be made in conformance to applicable codes and standards and will consider all of the required load cases and combinations (the cited SB report section reports seismic demands only, which are typically controlling). Concurrent loads are not considered; as such, the calculations presented are merely illustrative and are not final design calculations. During the detailed design phase, the combined license (COL) is required		***was used, but distributed differently than Mr. Wyllie had assumed. The Panel believes that a uniform distribution of bearing stress on the surface of the shear lug, as assumed by Mr. Wyllie, is quite conservative. More than likely the distribution is parabolic. However, conservatively assuming a triangular distribution of bearing stress, the resulting moment is [ ]^{a,c} Using this value and Mr. Wyllie's distribution of moment results in a plate bending stress of 50 ksi. Given the conservatism in each of the three calculations (staff, Wyllie, and the***

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				to ensure that WEC performs the design in accordance with the licensing basis methods, codes and standards and will include all applicable loads. However, to directly address the concern of high flexural stresses in the SC plate, staff performed an independent calculation which assumed the same applied moment [ ] a,c but distributed of the applied moment to each member end, rather than ½ each way as assumed by Mr. Wyllie. The staff's assumption is consistent with engineering textbook information on end actions and restrained members (Weaver, W., and G.M. Gere, <u>Matrix Analysis of Framed Structures</u>, 3rd Ed., Table B-1).	***Panel), the Panel considers the detail acceptable. This assessment, coupled with the staff's assertion that the actual detailed design of the connection will be in conformance with all applicable codes and standards and will consider all load cases, enables the Panel to consider that the follow-up is acceptable and the issue is resolved.***

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			Based on the revised demands, the results show that the stresses in the SC plate of concern are below yield. Based on the staff's assessment and the combined license (COL) requirement to ensure that detailed design is performed in accordance with the licensing basis methods, codes and standards, the staff considers this issue resolved.		
8	7.4.1	<u>PCS Tank, design-basis operating temperature</u> Page 86 of the DPO report states that, based on a discussion with WEC, the panel understands that the PCS tank heaters are only activated to ensure that the water temperature does not fall below 40°F. However, the panel did not understand the technical basis of the WEC assumption	Staff review finds that for the design of the PCS tank, WEC analyzed three design-basis thermal cases: (1) outside air at 115°F and tank water at 40°F; (2) outside air at -40°F and tank water at 40°F; and (3) outside air at 115°F and tank water at 70°F. During a June 2011 design audit (ADAMS Accession	**Staff considers this issue resolved.** Staff has performed a follow-up review of this issue and agrees with the panel's conclusion that the SB design continues to meet the requirements of GDC 2. Staff has performed an audit of the design of the PCS tank, including thermal analysis, and concluded that the design was adequate (June 2011).	***The Panel considers the follow-up to be acceptable and the issue to be resolved.*** ***This resolution is based on the staff's supplementary information from the WEC system specification document (SSD) for the tank that "WEC indicated that the average water***

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		that the tank water temperature at +40°F is conservative given that tank water temperature above +40°F resulted in an increase in the tank wall stress and reduced the tank's design margin.	No. ML112000530), staff requested WEC to determine the range of the water temperature in the tank under the "winter" thermal case where the air temperature outside the tank could be -40°F and the water heaters are triggered on and off. WEC indicated that the average water temperature is about 50°F plus or minus 2.5°F, based on the WEC system specification document (SSD) for the tank. The specification stated that if the tank water is at 50°F, the maximum percentage increase in the thermal gradient (delta T) is 13%. Staff review found that if the existing stress ratio in the design of the tank structure is conservatively increased by this value, the resulting stress-to-capacity ratio is still less than 1.0, and	The staff's evaluation is discussed in Section 3.8.4 of the Final Safety Evaluation Report (FSER) for AP1000. The staff considers the WEC assumption to use 50˚F for the water temperature of the PCS tank in the winter condition to have reasonable engineering basis. Given the small difference in results and the fact that there was no impact on the design, a revision of the DCD to reflect the 50˚F was judged to not be necessary during the audit. Staff continues to support this position. No further action is recommended.	***temperature is about 50 degrees F plus or minus 2.5 degrees F. The SSD states that if the tank water is at 50 degrees F, the maximum percentage increase in the thermal gradient (delta T) is 13%.***

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			thus the design is not affected. Based on this assessment for the increase in the stress ratio for all loads in the loading combination by the increase in the thermal gradient, which demonstrates that the design is still adequate, the staff considered the question raised to be adequately addressed. The details of the staff evaluation are in Section 3.8.4 of the Advanced Final Safety Evaluation Report (AFSER) for the AP1000 (ADAMS Accession No. ML112061231).		
9	7.4.2	PCS Tank: Consideration of uplift caused by the vertical component of the safe-shutdown earthquake (SSE) In the DPO report (on page 86), the panel concludes that a net vertical uplift of the PCS tank should be considered and that they	The method of analysis and design for the PCS tank is described in DCD Section 3.8.4.4.1. The Shield Building roof and the passive containment cooling water storage tank are analyzed using the three-dimensional finite-element quadrant model described in DCD	**Staff considers this issue resolved.** Staff has performed a follow-up review of this issue and agrees with the panel conclusion that the SB design continues to meet the requirements of GDC 2. Staff has performed an audit of both the analysis	***The Panel considers the follow-up to be acceptable and the issue to be resolved.*** ***This resolution is based on the staff's additional information that the detailing of the PCS tank wall and roof as***

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		could find no discussion on this issue for the tank design. The panel stated that WEC clarified that their computer analysis of the PCS tank considered appropriate hydrodynamic pressure caused by the vertical component of the earthquake (uplift) and that hydrostatic pressure is combined with the hydrodynamic pressure caused by horizontal and vertical components of the design-basis earthquake. However, the Panel did not review or verify the WEC computer analyses.	subsection 3G.2.3.1 with the ANSYS computer code. Loads and load combinations are given in subsection 3.8.4.3 and include construction, dead, live, thermal, wind, and seismic loads. The seismic response of the water in the tank is applied as static pressures corresponding to the impulsive and convective response. The results are used in the design of the tension ring, air inlet structure, PCS tank, Shield Building roof, and radial roof beams. The PCS tank is designed using the maximum accelerations at the applicable elevation resulting from time history dynamic analyses of the nuclear island. The tension ring and air inlet use maximum accelerations that are increased in such a way that the member forces in these regions envelop those from a response	and design of the PCS tank, including hydrodynamic effects, and concluded that the design was acceptable (June 2011; ADAMS Accession No. ML112000530). The staff's evaluation is discussed in Sections 3.7.2 and 3.8.4 of the FSER for AP1000. No further action is recommended.	***outlined is adequate to support net tension loads in the wall section and in the roof slab and that both the analysis and design detailing address uplift at the tank due to seismic loading.***

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				spectrum analysis using the refined NI05 model, as described in Appendix 3G.2.2.4. The staff performed an audit of the design of the SB PCS tank in June 2011 (ADAMS Accession No. ML112000530) and found that the design approach considered appropriate hydrodynamic pressures (convective and impulsive) caused by SSE demands (horizontal and vertical) and were consistent with those cited in the DCD. Staff observed that the convective (or sloshing) portion of the hydrodynamic pressure loading is quite small as compared to the inertial loading, which was applied in three orthogonal directions, explicitly considering vertical acceleration effects.	

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			Staff review also finds that the detailing of the PCS tank wall and roof is outlined so as to carry net tension in the wall section and in the roof slab. This observation indicates that both the analysis and the design detailing address uplift at the tank.		
10	7.4.3	<u>PCS Tank: Stability of the compression ring</u> The DPO panel (on page 86) identifies a concern (raised by Mr. Wyllie) with the load path of the SB compression ring under wind or seismic loads. Because the panel states that it did not review or verify the WEC computer analyses, they recommended that staff follow up on the issue.	The staff reviewed the concern of the lack of a mechanism in the SB roof design to resist twisting/warping of the compression ring. Specifically, Mr. Wyllie points to Figure 6.1-5 of the Design Report for the AP1000 Enhanced Shield Building (SB design report) highlighting the fact that the connection [ ] a,c	**Staff considers this issue resolved.** Staff has performed a followup review of this issue and agrees with the panel conclusion that the SB design continues to meet the requirements of GDC 2. On January 24, 2014, staff held a call with Mr. Wyllie to discuss this issue and a call summary was developed (ADAMS Accession No. ML14055A479). Staff reviewed the concern with Mr. Wyllie regarding unsymmetrical loads on the roof of the Shield Building,	***The Panel considers the follow-up to be acceptable and the issue to be resolved.*** **As the staff explains in their assessment, they followed-up with Mr. Wyllie on this issue. Staff showed that the W36 X 393 beams of concern to Dr. Wyllie are not part of the lateral force resisting system for the roof. Further, staff confirmed that a continuous load path across the vent**

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			lacks flexure rigidity. Mr. Wyllie further explains his concern that under asymmetrical loads, the roof could deform in such a manner that it would induce twisting in the compression ring; meanwhile, the roof design across the vent lacks the rigidity to resist such deformations. Mr. Wyllie highlights the possibility that the only structure appearing to resist this twisting is a complex load path around the compression ring. Lastly, Mr. Wyllie highlights the lack of deflection information for the compression ring to provide further insights into (supporting or contradicting) his concern, and he offers design changes to enhance the flexural rigidity across the roof vent. The staff confirmed its understanding of this concern during a teleconference call with	from wind, seismic, or aircraft impact and the significance of such a loading on the stability of the roof attributable to a perceived lack of structural continuity and strength across the roof vent area. The staff discussed the stability of the compression ring and the mechanism for load transfer across that area, which resolved Mr. Wyllie's concern and reassured him of the adequacy of the existing roof design. Mr. Wyllie said that the staff's explanations satisfied his concerns and that he had no further questions. No further action is recommended.	**area does exist and was shown by Westinghouse to have sufficient torsional rigidity to resist the design-basis loads. This assessment, coupled with the staff acceptance of WEC's commitment to adhering to the applicable codes for constructability, enable the Panel to find the follow-up acceptable and the issue to be resolved.**

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			Mr. Wyllie on January 24, 2014 (see ADAMS Accession No. ML14055A479). The staff reviewed Figure 6.1-5 and agrees that the [ ][a,c] are not detailed to provide flexure rigidity to the compression ring. However, the staff review finds that the [ ][a,c] Consequently, flexural detailing between those beams and the adjacent roof beams is not necessary to provide a load path for lateral loads. This detailing was also explicitly recognized in the roof analysis models		

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				developed by Westinghouse and audited by the staff. Further, staff confirmed that a continuous load path across the vent area, although different from the one envisioned by Mr. Wyllie, exists and was shown by Westinghouse, through detailed analyses, to have sufficient rigidity to resist the design-basis wind, tornado, and seismic loads (including asymmetrical load cases). Specifically, Figures 6.1-5 and D.2-3 depict a continuous steel ring beam that is rigidly connected to the radial roof beams. [ ] a,c is welded to the radial and ring steel beams. The concrete roof slab is connected to the steel beams through studs to form a composite steel-concrete section at the compression ring region. The concrete roof forms a membrane that is capable of resisting lateral	

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				loads through arch action. The concrete slab above the steel compression ring is enlarged to form a concrete embedded beam above the steel ring beam. The concrete embedded beam is detailed to have continuous hoop reinforcement and closed stirrups, both of which are integral in resisting twisting (torsion) stresses. These components form the mechanism, albeit complex as described in the letter, which is used by Westinghouse to resist twisting across the vent area. These components were judged by the staff to provide an adequate load path at the vent area based on the small deformation of the roof beams under seismic loads, the controlling design-basis load. On page H-172, the design report indicates the maximum roof beam deflection to be	

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				[ ][a,c] If the vent area had lacked structural continuity, instability would have been predicted and would have been indicated by much larger roof beam deflections. Regarding the concern raised in the letter regarding asymmetrical roof loads, the staff did not observe such large deformations under wind, snow, and seismic loads or during the aircraft impact assessment inspection. It is important to highlight that the staff is also relying on Westinghouse's commitment to codes (ACI 349 and AISC N690 in this instance) in the certified design, in ways consistent with the NRC's regulatory framework, to finalize and develop the construction details for the SB roof region, including appropriate rebar detailing and steel beam bracing to adequately resist the	

Item No.	DPO Report Section	Item	Staff Assessment	Conclusion	*Panel's Supplementary Evaluation*
			design-basis loads, including twisting stresses.		
11	7.4.4	PCS Tank: Ability of the structural details to resist shear failure The DPO panel (on page 87 of the DPO report) identifies a concern (raised by Mr. Wyllie) with the detailing of the base of the tank wall to the conical roof slab. The panel recommends that the detail be modified and recommended that staff follow up on this issue.	The staff reviewed the concern about the bend geometry of the horizontal bar at the base of the exterior tank wall and the potential to create a zone of no horizontal reinforcement (shear reinforcement). Specifically, Mr. Wyllie cites Figure 11.3-1 (on page 11-26) of the SB report and asserts that the radial horizontal bar at the intersection of the tank wall and the sloped roof will have an excessively large bend radius, thereby creating an area of no horizontal reinforcement between that horizontal bar and the first shear tie above it at elevation 293'-9". Mr. Wyllie based this conclusion on reading the size of this horizontal bar as a [.][a,c] Finally Mr. Wyllie proposed an improvement to the	**Staff considers this issue resolved.** Staff has performed a follow-up review of this issue and agrees with the panel conclusion that the SB design continues to meet the requirements of GDC2. On January 24, 2014, staff held a call with Mr. Wyllie to discuss this issue and a call summary was developed (ADAMS Accession No. ML14055A479). Staff reviewed the concern with Mr. Wyllie regarding the detailing of the connection between the PCS tank wall and the conical roof. The staff discussed additional information on an AP1000 design drawing (which contains proprietary information, and so is	***The Panel considers the follow-up to be acceptable and the issue is resolved. The following provides the Panel's basis for this resolution:*** ***The Panel's concern stemmed from the fact that Figure 11.3-1 of the SB report does not show the [ ][a,c] as would be required by ACI 349 Section 7.11.3 to anchor a stirrup. The staff explained to the Panel that Figure 11.3-1 is not a design or fabrication drawing and therefore does not show this level of reinforcement***

Item No.	DPO Report Section	Item	Staff Assessment	Conclusion	*Panel's Supplementary Evaluation*
			connection by replacing those horizontal [ ] a,c with similar [ ] a,c The staff confirmed its understanding of this concern during a telephone call with Mr. Wyllie. The staff reviewed Figure 11.3-1 and determined that the size of the bar in question is [,] a,c a, because there is pointer to the very bottom of that bar and the pointer reads [ ] a,c Another pointer on the figure which reads [ ] a,c and which may be seen as pointing to the bar in question (as well as the vertical rebar in the tank wall) might have been the source of the misunderstanding. Through the review of detailed SB roof calculation notes, the staff confirmed that the bar in question is a [ ] a,c which was selected by	non-public) which demonstrated to Mr. Wyllie that the current design of the reinforcing bars for that area is adequate and meets the intended purpose of the design. Mr. Wyllie stated that the staff's explanations satisfied his concerns and that he had no further questions. No further action is recommended.	***detailing. The staff and regional inspectors are well aware of this issue and committed to ensure that the anchoring and concrete cover requirements are met during construction.***

Item No.	DPO Report Section	Item	Staff Assessment	Conclusion	*Panel's Supplementary Evaluation*
				Westinghouse to have an appropriate bend radius to fit in the wall-roof joint geometry and preclude a region of unreinforced concrete at the base of the wall. Because the actual bar size [ ]a,c used in the design is consistent with the proposed detailing improvement by Mr. Wyllie, the staff finds that the concern is not substantiated. During the January 24, 2014, phone call with Mr. Wyllie, the staff also emphasized that the reinforcement detailing at this connection must satisfy the ACI 349-01 previsions, including the need to anchor shear ties around longitudinal reinforcement. As such, it is the staff's expectation and the code requirement for the [ ] a,c to be wrapped around the wall's horizontal hoop rebar.	

Item No.	DPO Report Section	Item	Staff Assessment	Conclusion	*Panel's Supplementary Evaluation*
			This expectation was also communicated to the regional staff to ensure appropriate implementation during construction.		
12	7.4.5	The DPO panel report identifies (on page 88) a concern (raised by Mr. Wyllie) with the potential for concrete voiding at the connection of the base of the tank wall to the conical roof slab. Regarding this issue, the DPO report notes that WEC stated to the Panel that (a) they appreciated the Panel's caution regarding the potential voids in the concrete and (b) they have discussed this issue with their constructor and are confident that the contractor will pour the self-consolidating concrete beneath the base of the tank and avoid any potential voids in the concrete underlying the tank.	Staff agrees with the concern for voiding in congested areas of the Shield Building. Staff (of Headquarters (HQ) and of Region II (RII)) is highly attuned to the need for adequate-quality concrete placement, especially in the highly congested areas of the nuclear island, including the Shield Building (as demonstrated by the numerous issues relating to detailed design identified by RII inspection staff). For the Shield Building constructability using self-consolidated concrete, the mockups are intended to demonstrate the viability of that product. Testing of	The concrete quality issue is not specific to the DPO and is of great importance to the staff and the licensees. Mockups are under construction and are being tested. The staff is awaiting the licensees' mockup testing results and will engage through the appropriate inspection channels. No further action in association with the DPO is recommended on this topic.	***The Panel considers the follow-up to be acceptable and the issue to be resolved.*** ***This resolution is based on the staff's commitment that for the shield building constructability using self-consolidating concrete, the mockups will demonstrate the viability of the process and that the testing of the mockups will be inspected by NRC inspectors.***

Item No.	DPO Report Section	Item	Staff Assessment	Conclusion	*Panel's Supplementary Evaluation*
		The Panel recommends that the staff follow up on this issue.	the mockups for defects and voids is an activity that is planned to be inspected by the inspectors and assisted by NRO staff.		
13	7.4.6	<u>PCS Tank: Seismic Margin and HCLPF</u> The DPO report states (on page 88) that the Shield Building reanalysis in Appendix L shows the PCS tank wall design margin is reduced from [ ] a,c The Panel could not locate the HCLPF calculation based on this design margin [ ] a,c The Panel recommends that the staff follow up on this issue.	See Item 1, above.	**Staff considers this issue resolved.** Staff has performed a follow-up review of this issue and concludes that there is no requirement for WEC to include temperature effects in the calculation of DCD HCLPF values. Accordingly, the DCD HCLPF values for the Shield Building (in Table 19.55) remain acceptable. This position is consistent with the DPO panel's conclusion (on page 6 of the DPO report) that the AP1000 SB does meet the NRC seismic margin requirements and is also consistent with ASME/ANS RA-S-2009 recommendations not to include thermal stresses in SMA calculations.	***See evaluation for Item 1***

Item No.	DPO Report Section	Item	Staff Assessment	Conclusion	*Panel's Supplementary Evaluation*
				No further action is recommended.	
14	7.5	Staff's Seismic Evaluation in FSER Section 3.7 The Panel did not find any reasons for not providing any evaluation of these criteria or any reference to sections in which such evaluations are provided. 3.7.2.II.9. Effects of Parameter Variations on Floor Response Spectra 3.7.2.II.10. Use of Equivalent Vertical Static Factors 3.7.2.II.11. Methods Used to Account for Torsional Effects 3.7.2.II.12. Comparison of Responses 3.7.2.II.13. Analysis Procedure for Damping 3.7.2.II.14. Determination of Seismic Overturning Moments and Sliding Forces The Panel recommends that the staff follow up on this issue to determine whether	Staff notes that the list of Section 3.7 SRP review areas identified in the DPO report were reviewed by staff for Revision 15 of the AP1000 DCD. The staff's evaluation of each listed item is described in the Final Safety Evaluation Report Related to Certification of the AP1000 Standard Design (NUREG-1793, Initial Report; September 2004, ADAMS Accession No. ML060330557). Because the amendment to Revision 19 of the AP1000 DCD did not make changes/revisions to these application areas, staff did not perform a re-review in accordance with design finality. The applicant did not change these sections of the application and, therefore,	**Staff considers this issue resolved.** The missing SER areas identified by the panel were part of the original SER for the DCD Rev. 15 certification and were not restated in the SER amendment, which is consistent with the design's finality. No further action is recommended.	***The DPO Panel could not confirm the staff's basis for not performing and documenting the seismic evaluation of DCD sections 3.7.2.11 and 3.7.2.12 in the SER supplement for certified AP1000 design. (See enclosure 10 for additional details of the Panel's evaluation.)***

Item No.	DPO Report Section	Item	Staff Assessment	Conclusion	*Panel's Supplementary Evaluation*
		the seismic analyses of the certified AP1000 design have been evaluated and documented in the FSER for the Standard Review Plan (SRP) acceptance criteria 3.7.2.II.9 through 3.7.2.II14.	the staff's conclusions on these sections did not change and were not reiterated in the SER supplement.		
15	5.2	In Section 5.2 of the DPO Report, the panel concluded that it was not able to determine whether WEC had met the intent of subsection 2.4.1(4) of NEI 07-13 with regards to benchmarking of computer models for AIA.	See Enclosure 4.	See Enclosure 4.	***The Panel considers the follow-up to be acceptable and the issue to be resolved. The bases for this resolution are described in Enclosure 4.***
16	7.0-B.1	The panel made an observation that peer reviews of the staff's safety evaluations would improve the quality and efficiency of the reviews and could potentially reduce non-concurrences and DPOs.	In the context of the AP1000 Shield Building review, a peer-review process was used. The technical review was conducted by several members of the staff and contractors from Brookhaven National Laboratory. Traditionally, in the structural engineering branch, the staff and contractors engage in technical	The staff reviewed this observation and finds it not to impact the staff's conclusions regarding the safety of the Shield Building. Regarding more recent reviews, the peer review of technical documents is an integral part of the license-amendment safety-evaluation process.	***The Panel agrees that no further action is recommended regarding the DPO. The recommendation was meant to considered in the broader sense and has been adequately addressed by the staff.***

Item No.	DPO Report Section	Item	Staff Assessment	Conclusion	***Panel's Supplementary Evaluation***
				discussions on safety issues and agree on the technical positions documented in the staff's FSER. The staff used this process during the review of the AP1000 Shield Building. In addition to the above, the staff requested the Office of Regulatory Research to perform a peer review of the design using outside expertise. Four leading experts peer-reviewed the building design and provided their recommendations. This independent review is documented in a RES summary evaluation report (ADAMS Accession No. ML103080105). More recently, the staff has been employing a team-review approach, in which a team of staff reviews the technical issues and guides the resolution. A staff member then develops the safety evaluation, seeking input and insights	No further action is recommended.

Item No.	**DPO Report Section**	**Item**	**Staff Assessment**	**Conclusion**	***Panel's Supplementary Evaluation***
			from the team members. The branch chief reviews and concurs on the final product. As such, peer reviews are currently an integral part of the structural engineering branch reviews.		
17	7.0-B.2	Supplemental AIA for the PCS tank and containment was not inspected by the staff.	See Item 4 above.	See Item 4 above.	***See evaluation for Item 4***
18	7.0-B.3	Conflicting HCLPF values in two parts of the application.	See Item 2 above.	See Item 2 above.	***See evaluation for Item 2***
19	7.0-B.4	HCLPF values did not consider thermal impacts.	See Item 1 above.	See Item 1 above.	***See evaluation for Item 1***

Enclosure 3. NRO/DE Evaluation of the Effects of Cold Temperatures on AP1000 Shield Building Aircraft Impact Assessment and the Supplementary Assessment by DPO panel Member [redacted] (Item3, Enclosure 2)

This enclosure provides the staff's evaluation of cold-temperature effects on the AP1000 Shield Building (SB) aircraft impact assessment (AIA).

Key Message

The Differing Professional Opinion (DPO) Panel report (on page 82) recommends that staff follow up on issues raised by panel member [redacted] in report Section 4.2.3.5.1(b), which relate to consideration of bounding ambient temperature in conjunction with AIA. Panel member [redacted] concern, which coincides with a similar concern by the DPO submitter, is that the ability of the Shield Building wall to resist aircraft impact under cold weather has not been demonstrated and is doubtful (page 40).

Staff has performed a follow-up review of this issue and continues to believe that the AP1000 Shield Building AIA was performed in accordance with regulatory guidance and therefore satisfies the regulations. Staff considers this issue resolved based on the evaluation discussed below.

Staff Evaluation

The staff understands that the concern relates to performing the AIA on the Shield Building with material models adjusted to reflect the material behavior under the bounding cold ambient temperature (-40˚F in the case of AP1000). Although the majority of the DPO panel did not find it necessary to consider the bounding ambient temperature in combination with AIA, the staff investigated the matter to assess its safety merits. The staff has reviewed the relevant regulatory requirements, implementing guidance, and technical basis concerning cold-weather effects on the AP1000 Shield Building. The staff's findings are described below.

<u>Regulatory Basis</u>

The proposed analysis does not appear to be consistent with the regulatory requirements for AIA and with the NRC's treatment of other beyond-design-basis events.

As noted in the DPO report (on page 26), the aircraft impact rule (Section 50.150, "Aircraft Impact Assessment," of Title 10, "Energy," of the *Code of Federal Regulations* (10 CFR 50.150)) requires an applicant to perform a design-specific assessment of the effects on the facility of the impact of a large commercial aircraft. The rule requires that, using realistic analyses, the applicant shall identify and incorporate in the design those design features and functional capabilities to show that, with reduced use of operator actions, (a) the reactor core remains cooled or (b) the containment remains intact and spent fuel cooling and spent fuel pool integrity is maintained.

The term "realistic" is incorporated in the rule to convey a certain framework for the AIA. The AIA rule's Statement of Consideration (SOC) clarifies that "realistic" is a relative term and is simply intended to avoid requiring a designer to use conservative or bounding assumptions in recognition of the NRC's determination that the impact of a large commercial aircraft is a beyond-design-basis event (refer to page 27 of the DPO report). The rule language and the SOC clarification are consistent with standard engineering practice and with the NRC's treatment of beyond-design-basis events.

Design-Basis Approach

In standard engineering practice, distinctions are made regarding the analysis approach of design-basis and beyond-design-basis loadings. For design-basis loadings, such as extreme seismic and tornado events, the practice is to use conservative assumptions such as code-specified material properties and linear response analyses. Staff notes that in structural assessments of the combined effects of the safe-shutdown earthquake (SSE) or tornado events with extreme ambient or accident temperatures, the induced material stresses caused by each loading regime are combined (within appropriate combinations), but the material models used in the analysis are not corrected to reflect temperature effects. Such approximations are not considered large enough to question the validity of the analysis results, but rather they indicate that additional refinement and precision have little to no effect on the conclusions and are not necessary to reach reasonable assurance of the expected behavior.

Beyond-Design-Basis Approach

The practice for beyond-design-basis events, such as the review-level earthquake, is to rely on best-estimate parameters (e.g., median-value fragilities) to avoid excessive conservatism. That is, the beyond-design-basis event is considered the extreme parameter, while other parameters in the assessment remain at their best-estimate values. The premise is to use sound engineering with reasonable, not unwarranted, conservatism.

The class of events considered for beyond-design-basis assessments in nuclear power plants (NPPs), including AIA, contains considerable complexity and associated uncertainties. Recognizing the complexity and the level of uncertainty associated with AIA, the agency adopted, as part of the "realistic" approach, an analysis using simplified loading parameters (prescribed force-time curves instead of an explicit aircraft model) and rule sets (structure/fire/shock). The AIA parameters provided by the NRC to NPP applicants are not representative of a particular bounding aircraft (speed, model, fuel load, etc.), but rather they are based on realistic assumptions built on actual impact events and typical fuel loads for commercial aircraft. Other uncertainties are also associated with the structural aspect of the AIA, such as the angle and location of impact and the effect of debris generated on impact. And in terms of fire and explosions associated with AIA, substantial uncertainty surrounds fire-induced spurious actuations.

Based on the above examples, staff believes the recommendation to use lower-bound material properties (i.e., corrected for ambient thermal) is inconsistent with current engineering approaches to civil engineering problems and is not supported by industry codes and standards. The agency's risk-informed regulatory approach avoids the use of unwarranted conservatism, while on the other hand it provides assurance of safety through the use of a defense-in-depth approach. This policy plays an important role in determining the necessary degree of precision in engineering assessments.

In a global sense, such complexity and uncertainties associated with AIA are acknowledged and addressed by the agency through the defense-in-depth inherent in the combined regulatory requirements of 10 CFR 50.150 and 10 CFR 50.54(hh)(2). 10 CFR 50.54(hh)(2) requires all plants to develop and adopt mitigation strategies to address loss of large areas of the plant because of fire or explosions. Both regulations are intended to enhance a NPP's ability to withstand the impacts of a large fire or explosion, whether it is caused by an aircraft impact or other events.

Given the scope of uncertainties, the degree of conservatism in the implementation guidance (as discussed below), and the defense-in-depth inherent in the regulatory framework, an increased precision in treating ambient temperature, beyond that customarily followed in civil engineering problems, would have little to no effect on the AIA's overall accuracy.

Implementing Guidance

The approach proposed by panel member [redacted] does not appear to be consistent with the NRC's regulatory guidance for AIA.

NRC Regulatory Guide 1.217, "Guidance for the Assessment of Beyond-Design-Basis Aircraft Impacts," provides acceptable methods for an applicant to meet the AIA rule. This Regulatory Guide endorses Nuclear Energy Institute (NEI) 07-13, which the industry experts developed to serve as a methodology for assessing NPPs for the effects of aircraft impact. NRC staff participated extensively in the development of NEI 07-13.

A review of NEI 07-13 indicates that the goal of developing a realistic framework for AIA was at the core of the method and guided many of the decisions. Of interest to this discussion are the acceptable material properties identified in NEI 07-13 for use in the AIA. These properties are best-estimate values adjusted for aging and strain-rate effects. Room-temperature material properties are assumed because it is not expected that temperature effects (including cold weather) significantly reduce capacity for strain rates expected for AIA. In fact, these material properties and the underlying assumption are consistent with how the industry's codes and standards treat impact problems under other events. As an example, for design-basis analyses of events involving tornado and hurricane missiles where conservative assumptions are used, the code provisions in ACI 349 and AISC N690, including empirical capacity equations, are based on room-temperature material properties, and are not adjusted for bounding ambient or accident temperatures. Once again, the premise embodied in these codes and standards and adopted in the NRC's guidance on AIA is that, in a global sense, the inherent conservatism and the goal to make these methods feasible yet realistic outweighs the benefit of further precision in one or more variables.

Staff also notes that the assumption to use best-estimate room-temperature material properties is consistent with the analyses performed for the U.S. NPP operating fleet in 2002 through 2007 by both the NRC and the industry (NEI and the Electric Power Research Institute (EPRI)).

It is also worth noting that in design-basis threat (DBT) studies under explosives (both underwater and air-blast); nominal material properties rather than bounding properties are used.

Technical Basis for Cold-Temperature Effects on AIA

The nil-ductility transition (NDT) for A572 steel under an AIA regime is likely much less than 30°F and reasonably lower than the mean cold temperature in the entire United States on any day.

Panel member [redacted] refers to a DPO report figure (in Section 4.2.3.5.1) which presents fracture toughness as a function of strain rate and temperature. The figure represent fracture-toughness data at three loading strain rates, dynamic (10 sec^{-1}), intermediate (10^{-3} sec^{-1}), and, slow (10^{-5} sec^{-1}). The fracture-toughness curve labeled “dynamic” has an NDT of approximately 30°F. Panel member [redacted] low-temperature concern stems from the assumption that the “dynamic” strain rate curve (10 sec^{-1}) is similar to the strain rate of an aircraft impact. Based on two independent sources, as described below, that rate is too high for aircraft impact problems and a more appropriate strain rate up to an order of magnitude lower.

To guide analysts performing dynamic analysis of concrete structures, LS-DYNA (the finite-element analysis (FEA) code that WEC used to perform the Shield Building AIA) provides information on the expected strain-rate regimes for a range of problem types. Figure 2, below, reproduced from the LS-DYNA user manual, indicates that the expected strain rate regime for an aircraft impact is less than 1.0 sec^{-1} and significantly less than the regime cited by [redacted] as cause for concern (10 sec^{-1}).

NEI 07-13 also provides a different insight into the strain rate. For a dynamic analysis of structures, it is typical for engineers to assign dynamic increase factors (DIFs) to material properties to account for enhanced capacities resulting from strain-rate effects. NEI 07-13 recommends a DIF for concrete elements of 1.25 on the ultimate compressive strength. For the ultimate strength of steel elements, the guidance recommends DIFs of 1.05 for reinforcing steel and 1.10 for carbon steel plates. Staff reviewed the basis for the NEI 07-13 DIFs (i.e., U.S. Army TM-855, “Fundamentals of Protective Design for Conventional Weapons”) to determine the strain rates corresponding to the cited DIFs. The range of DIFs from 1.05 to 1.25 corresponds to strain rates of 0.10 sec^{-1} to 0.50 sec^{-1} for both steel and concrete materials. These strain rates are well within the range indicated in the LS-DYNA manual for expected aircraft impact strain rates (<1.0 sec^{-1}). Accordingly, while an aircraft impact on a massive concrete structure like the Shield Building is a highly dynamic event, the strain rates involved are significantly lower than 10 sec^{-1}.

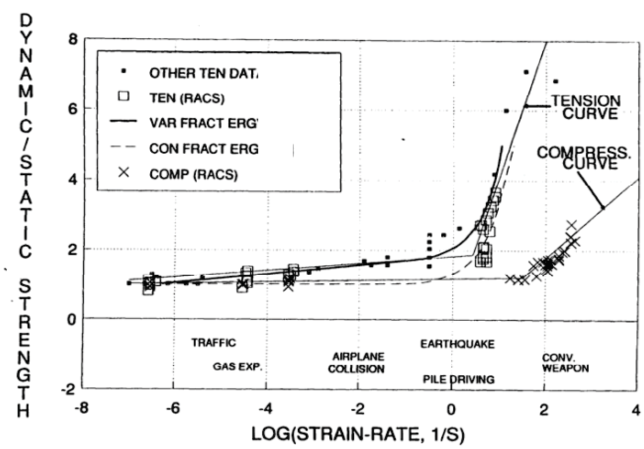

Figure 2. Figure from the LS-DYNA Users Manual Indicating Strain Rates for Various Dynamic Problems on Concrete Material.

To calculate a more representative NDT temperature for the AIA strain rate, staff performed a simplified assessment of the impact of cold temperature on fracture toughness at the strain rate range expected for AIA, conservatively estimated to be [.][a,c] Because the staff did not have A572 Grade 50 fracture-toughness data corresponding to this strain rate, a pragmatic approach was used. By plotting NDT temperature versus strain rate for A572 shown on the DPO report figure in Section 4.2.3.5.1 and interpolating for the values of [,] a,c staff estimates the NDT temperature to be in the range of [ ] a,c (see Figure 3). Even with this conservative assumption, which is based on plane strain test data, the interpolated NDT temperature is significantly[ ] a,c the 30°F NDT temperature (cited in the DPO report) and reasonably [ ] a,c the average (best-estimate) cold temperature in the entire United States on any given day.

Staff also concurs with the majority of the DPO panel that A572 material is a common engineering material used in U.S. civil engineering infrastructure (bridges, buildings, etc.) in all parts of the county. Brittle fractures of this material caused by cold weather effects have not been cited as a concern. This observation further suggests that the [ ] a,c

a,c

Recommendation for Follow-up

Based on its regulatory and technical review of the concern, the staff agrees with the DPO report position that the AP1000 Shield Building is competent to stop a large commercial aircraft without perforation of the wall or threatening the integrity of the containment (page 35). Further, staff agrees with the DPO panel's majority position (Hackett/[redacted]) that -40°F is a lower-bound temperature and results in lower-bound material properties. The use of such lower-bound properties is not in conformance to the AIA rule, the SOC of the AIA rule and implementing guidance, industry codes and standards for impact problems, and civil engineering practice for both design-basis and beyond-design-basis events. Further, recognizing the scope of uncertainty in AIA and the defense-in-depth in the regulations, further precision in one parameter (ambient temperature) would have little to no effect on increasing the overall accuracy of the assessment. Moreover, considering the best-estimate cold temperature in the United States is likely to have little to no effect on the Shield Building AIA analysis results considering the likelihood that A572 NDT is much less than 30°F for aircraft impact strain rates.

Supplementary Assessment of DPO Panel Member [redacted] Regarding the Effect of Cold Temperatures on the Shield Building Aircraft Impact Assessment (AIA)

<u>Issue:</u>

Panel member [redacted] expressed concern that the ability of the Shield Building wall to resist aircraft impact under cold weather has not been demonstrated and is doubtful.

<u>Information reviewed</u>

Panel member [redacted] reviewed the pertinent information and bases that the staff provided in their July 24, 2014 report 'Response to the Report of the Ad Hoc Panel for DPO 2012-002', and met with the RES analysts (Jose' Pires and Hernando Candra) who had performed an independent finite element non-linear dynamic analyses of the AP1000 shield building for the effects of an aircraft impact to support the AP1000 certification.

The primary basis for Panel member [redacted] low-temperature concern is whether the dynamic strain rate curve (10 sec^{-1}) is applicable to the aircraft impact analysis as one would expect high strain rate in the shield wall steel plates due to an aircraft impact loading. The application of the dynamic strain rate curve (10 sec^{-1}) could potentially predict lower fracture toughness strength of the shield wall steel plates than assumed in the certified AIA analysis that was performed at room temperature.

The staff in Enclosure 3 to the July 24, 2014 report, under 'Technical Basis for Cold-Temperature Effects on AIA' stated:

> ***"To guide analysts performing dynamic analysis of concrete structures, LS-DYNA (the finite-element analysis (FEA) code that WEC used to perform the Shield Building AIA) provides information on the expected strain-rate regimes for a range of problem types. Figure 2, below, reproduced from the LS-DYNA user manual, indicates that the expected strain rate regime for an aircraft impact is less than 1.0 sec^{-1} and significantly less than the regime cited by [redacted] as cause for concern (10 sec^{-1})."***

Panel member [redacted] reviewed strain rates from RES independent AIA analysis of the AP1000 shield building and observed that:

1. ***RES analysts agree that Figure 2 (On Page 33 of the Staff's July 24, 2014, report) does not appear reasonable as it shows the strain rates due to earthquake higher than that due to airplane collision. Further, the figure is shown to be applicable to "concrete problem", and not to steel plates.***
2. ***RES analysts stated that the strain rate specified on second paragraph of the Staff's report on Page 33, "expected strain rate regime for an aircraft impact is less than 1.0/sec." is not correct. RES stated that the correct value of the strain***

rate to be [ ] a,c RES indicated that they have provided their comments to Joe Williams (8/28/2014) to reflect the correction in the meeting summary report.

3. *RES analysts, from the review of the detailed results of the AIA computer analysis showed a strain rate [ ] a,c from a selected analysis output for steel plate strains.*

Based on the review of the RES computer analysis results for AIA, Panel member [redacted] concludes that (1) Figure 2 in the report does not appear reasonable and applicable to steel plates (2) staff's assertion that "the expected strain rate regime for an aircraft impact is less than 1.0 sec^{-1}" is not supported by the RES analysis results, and (3) the strain rate is predicted [ ] a,c

Panel member [redacted] finds from Figure 3 'Simplified Assessment of Strain Rate' in the report that NDT temperature is about 10°F for A572 Steel at strain rate of [ ] a,c The best estimate (mean) of outside cold temperatures in about one-third of the United States in winter months (December, January, and February) is above the NDT of 10°F. On this basis it is concluded that the fracture toughness strength and ductile behavior of the shield wall steel plates due to an aircraft impact are not anticipated to be affected during winter months in the United States.

DPO Panel Member [redacted] Conclusion:

Based on the review of the staff's supporting information in the report and the RES independent AIA analysis of the AP1000 shield building, Panel member [redacted] concludes that the fracture toughness strength and ductile behavior of the shield wall steel plates due to an aircraft impact are not anticipated to be affected during winter months in the United States. The Shield Building wall is expected to resist aircraft impact under cold weather in the United States.

Enclosure 4 - NRO/DE Evaluation of Aircraft Impact Assessment Benchmarking and the DPO Panel's Supplementary Assessment (item 15, Enclosure 2)

This enclosure addresses the Differing Professional Opinion (DPO) panel comments on AP1000 Shield Building aircraft impact assessment (AIA) benchmarking.

In Section 5.2 of the DPO Report, the panel concluded that it was not able to determine whether WEC had met the intent of subsection 2.4.1(4) of NEI 07-13 with regards to benchmarking of computer models for AIA. Subsection 2.4.1(4) of NEI 07-13 states that past experience with aircraft-impact analysis of nuclear power-plant structures has not been all-inclusive and that new plant designs may contain design features for which experimental and analytical experience is lacking. The guidance states that, in these cases, it is important to recognize that these new design features may be subject to failure modes that are outside the existing experience base and may require experimentally verified analytical evaluations.

Staff has performed a followup review of this concern and understands the issue of analytical model validation and its importance. It should be noted that, because the NRC technical reviewers participated in the collaborative development of NEI 07-13 analysis guidance, the issue of analytical model validation was fully recognized during the review and inspections of the WEC AIA. It should also be noted, however, that to be consistent with the AIA rule, the WEC AIA (including analyses and benchmarking) is not submitted for staff review, but is rather subject to inspection.

Between September 27 and October 1, 2010, staff performed an inspection of the WEC AIA and specifically focused on the issue of concern (i.e., analytical model validation). To support the staff, a structural engineering expert from Sandia National Laboratory who was familiar with the operating fleet's vulnerability assessments helped to review the AIA of the Shield Building. The inspection report (ADAMS Accession No. ML102980583) describes the staff's review of the AIA to ensure consistency with subsection 2.4.1(4) of NEI 07-13 as follows:

> "*The NRC inspection team reviewed the WEC AP1000 AIA structural damage assessment including design inputs, analysis parameters and assumptions, computer codes, method used for structural analyses, and results. Specifically, the NRC inspection team reviewed the LS-DYNA computer code used in the structural analysis for the AP1000 AIA to determine if the applicant had adequately validated and verified the code for the applicable class of problems assessed and had adequately documented the validation and verification. Section 2.4.1 of NEI 07-13 states that "new design features may be subject to failure modes that are outside of their existing experience base, and may require experimentally-verified analytical evaluation" or benchmarking."*
>
> *"The AP1000 Shield Building makes use of steel concrete composite construction for which there is greater uncertainty with respect to impact behavior compared to reinforced concrete. WEC performed benchmarking of the LS-DYNA analysis code on steel concrete structures using the Winfrith concrete model. The NRC inspection team reviewed the benchmarking process and the technical justification and verified it to be accurate and complete.*"

During subsequent inspections (in May 2011 through August 2011; see ADAMS Accession No. ML112650748), the staff re-examined WEC's benchmarking and verified that the [

] [a,c] Staff also notes that other independent analysis models, performed by the U.S. Nuclear Regulatory Commission's Office of Nuclear Regulatory Research (NRC/RES) and Dr. Rashid (the contractor hired by the panel), have undergone their own independent benchmarking and have arrived at the same overall conclusion (no perforation of the SB). The staff views these findings by the independent analysts as another element of reasonable assurance in the adequacy of the WEC AIA. Based on the NRC inspection findings, the staff continues to believe that the criteria of subsection 2.4.1(4) of NEI 07-13 have been satisfied and that the AP1000 Shield Building AIA was performed in accordance with regulatory guidance and therefore satisfies the regulations. Staff considers this issue resolved. No further actions are recommended.

DPO Panel's Supplementary Assessment of AIA Benchmarking

Based on the DPO Panel's interviews with staff and their consultant, the Panel believed that the staff's inspection and audit of the WEC AIA calculations was entirely focused on review of modeling assumptions, material models and analysis parameters, and concerns related to time step size and mesh refinement. The Panel was not aware that one aspect of the staff's inspection of the AIA calculations focused on analytical model validation and benchmarking.

The purpose of analytical model validation and benchmarking is to provide assurance that the computer code can adequately model the SB wall's structural behavior to simulate the response due to aircraft impact. The staff's inspections of WEC's model validation and benchmarking provide the Panel with additional assurance that the computer code can adequately model the SB wall's structural behavior to simulate the response due to aircraft impact. To further assure that the WEC analytical model validation and benchmarking efforts resulted in a quality AIA, the NRC staff, using the same computer code as WEC, performed an independent detailed AIA and confirmed the WEC result that the aircraft impact does not result in perforation of the SB wall. In addition, the DPO Panel's aircraft impact expert, using a different computer code, confirmed the same result (See DPO Panel Report pages 34 and 35.).

Enclosure 5 - NRO/DE Evaluation of Submitter's Response (May 1, 2014) to the DPO Panel Report and the Panel's Supplementary Evaluation

This enclosure provides the staff's evaluation of the submitter's response to the Differing Professional Opinion (DPO) panel report.

On May 1, 2014, the DPO submitter sent an email (with attachment) to the NRO Office Director and the DPO panel; this email provided feedback on the DPO panel report.

As requested by Office of New Reactors (NRO) management, Division of Engineering (DE) staff performed a review of the email and supporting documentation to identify any new safety concerns or insights that could impact the staff's conclusions on the safety of the Shield Building design and would require further actions. The staff's review of the email and attachment identified no new safety concerns or insights. The staff's review finds that the submitters' email and attachment do not raise new issues that have not been addressed during the non-concurrence, DPO, or DPO followup actions by the staff. Further, the email does not alter the staff's conclusions regarding the acceptability of the Shield Building (SB) design. The basis for this finding is described below.

The submitter's primary concerns identified in his feedback relate to passive containment cooling system (PCS) tank anchorage, in-plane shear design, out-of-plane shear design, and Shield Building aircraft impact assessment (AIA), including comparisons to other standard designs (such as EPR).

PCS Tank Anchorage

PCS tank anchorage is addressed in Item 11 of Enclosure 2. This issue was raised by the DPO panel consultant Mr. Wyllie and was discussed with him in a phone conversation. After his conversation with the staff, Mr. Wyllie had no further concerns with this item.

In-Plane Shear Design

In his email and attachment, the submitter cites a 1999 American Concrete Institute (ACI) report (ACI 445R-99) and National Earthquake Hazards Reduction Program (NEHRP) guidance which indicate in-plane shear strengths less than $10\sqrt{f_c'}$ under certain conditions. Staff review of the information finds that while it contains useful research and insights, the information has not reached the level of consensus to be codified in the latest building codes, which incorporate the state-of-the-art knowledge on the subject. This is evidenced by the observation that both ACI 349-01 and ACI 318-11, published after the ACI 445R report, contain shear wall provisions with a limit value of $10\sqrt{f_c'}$. In addition, the actual capacity of the AP1000 Shield Building's steel concrete composite (SC) walls was shown to be significantly greater than the ACI code capacity limits (see Figure 4 below). This is because the steel plates can carry large tension and compressive forces needed to resist in-plane shear. As such, the submitter's concerns related to the in-plane shear capacity of the AP1000 Shield Building's SC walls are of low safety significance.

Staff notes that the issue of in-plane shear capacity has been extensively discussed in the 2010 SB non-concurrence (NC) (ADAMS Accession No. ML103370648), the staff response to the NC (ADAMS Accession No. ML103020239), and the DPO panel report). This is not a new issue and the DPO panel addressed it thoroughly in its report.

Out-of-Plane Shear Design

In Section 3.0 of the response to the DPO panel report, the submitter claims that the DPO panel report's statement (in Section 2.2) that all four of the consultants engaged by the Office of Nuclear Regulatory Research (RES) also agreed that the out-of-plane shear demand-to-capacity ratio was acceptable with sufficient margin is false and misleading. Staff notes that RES documented the positions of the consultants in a summary evaluation report (ADAMS Accession No. ML103080105). The RES report states (on page 38) that all consultants agreed that the SB demand-to-capacity ratios were acceptable with sufficient margin and it also describes the position of Dr. Vecchio.

The RES report states that while Dr. Vecchio indicated the need for additional testing and tie-bar spacing shorter than [ ] [a,c] he recognized that ductility requirements in ACI 349 (Chapter 21) relate primarily to frames and not to walls or the Shield Building structure. This expert also understood that ductility detailing requirements in ACI 318 (Ch. 21) relate to structures for which it is allowed to reduce seismic forces by relying on ductile structural response, which is not the case for the AP 1000 Shield Building structure. Recognizing these two aspects, Dr. Vecchio preferred to apply ductility detailing requirements to every region of the Shield Building in the event of unforeseen circumstances in which the loads acting on the structure are of much larger magnitude than anticipated.

Other RES consultants agreed that given (1) the use of close tie-bar spacing in regions of high out-of-plane shear demand and (2) the low demand-to-capacity ratios presented in the WEC report for those portions of the wall with [ ] [a,c] tie-bar spacing, it is not appropriate to use the ACI 349 and ACI 318 codes as the basis for insisting on ductility requirements for out-of-plane shear. Consequently, additional out-of-plane shear reinforcement for the SC wall modules in these portions of the wall is considered unnecessary (Reference Figure 5 below). RES and NRO staff agreed with this conclusion.

Shield Building ductility (or lack thereof) was the main concern raised in the non-concurrence on the Shield Building and was addressed extensively by the staff. Based on the AP1000 design-specific tests, the analyses performed by the applicant, confirmatory analyses, and the margin inherent in the design, the staff concluded that the Shield Building possessed adequate ductility and satisfied applicable design code provisions. The staff's evaluation of the SB tie-bar spacing and consideration of system ductility is described in Section 3.8.4 of the Advanced Final Safety Evaluation Report (AFSER) for AP1000 (ADAMS Accession No. ML112061231).

Shield Building AIA

The submitter's email makes a comparison of AIA results for the AP1000 SB and another nuclear power plant design (EPR). This issue is also discussed in the submitter's DPO and addressed in the DPO panel report (in Section 4.2.3). The submitter's concerns of inadequate wall thickness, based on other certified designs that have been or are under review, have been raised before by the submitter. The staff position is that this approach is an oversimplification that neglects various significant aspects that influence the behavior of a wall subject to an impact load, such as reinforcement ratio, wall span/radius, and curvature. The AP1000 Shield Building wall thickness has been verified by three independent AIAs to be adequate.

a,c

a,c

Figure 5. AP1000 SB wall testing results (out-of-plane); Source; Staff presentation to ACRS Full Committee; December 2, 2010.

DPO Panel's Supplementary Evaluation of the Submitter's email of May 1, 2014.

<u>Comment 1:</u>

The DPO Panel agrees with the staff's evaluation of the Submitter's email of May 1, 2014 contained in Enclosure 5.

<u>Comment 2:</u>

On page 1 of the email, Item I the Submitter states,

> ***"The DPO Panel made an independent calculation and concluded that the PCS tank anchorage could shear-off."***

This Item is addressed in Comment 5 of the DPO Panel's response to the Submitter's email of March 27, 2014 in Enclosure 7.

<u>Comment 3:</u>

On page 1 of the email Item II the Submitter states,

> ***"The AP1000 shield building brittle wall is inadequate to resist aircraft impact."***

This Item is addressed in the DPO Panel's response in Comment 2 in Enclosure 7 and the DPO Panel Report Sections 4.2.3, 4.2.3.1 through 4.2.3.4 and 4.2.3.5.3.

<u>Comment 4:</u>

On page 3 of the email Item III the Submitter states,

"The AP1000 shield building wall does not meet GDC2 and seismic margin analysis."

This Item is addressed in DPO Panel Report Sections 4.1.2.4, 4.1.4 and 4.4.

Enclosure 6 - NRO/DE Response to Submitter's May 28, 2014, Email Comments on Curvature Effects and the DPO Panel's Supplementary Evaluation

On May 28, 2014, the Differing Professional Opinion (DPO) submitter sent an email to the director of the Office of New Reactors (NRO) and to the DPO panel which questioned the DPO panel report's position (on page 33) stating the beneficial effects of Shield Building wall curvature in resisting aircraft impact scenarios.

As requested by NRO management, Division of Engineering (DE) staff performed a review of the email and supporting documentation. The staff's review of the email and cited journal article identified no new safety concerns or insights. Further, staff finds that the cited article does not negate the DPO panel report's conclusions regarding the beneficial effects of curvature and does not alter the staff's conclusions regarding the acceptability of the Shield Building (SB) design. The basis for this finding is described below.

The email cites a recent article in the May-June 2014 issue of the American Concrete Institute (ACI) Structural Journal. The email asserts that the subject ACI paper contradicts the curvature effect on shear strength in of the AP1000 Shield Building (SB) wall as described in the DPO panel report. Staff located the ACI article in the NRC library so that a diligent review could be performed and the full context of the information cited in the email could be understood.

The ACI article describes the results of nine bending tests performed on arch-shaped members without transverse reinforcement. The goal of the testing was to represent the behavior in a reinforced structure governed by beam shear-carrying action rather than by arching action. The shear capacities of the arched specimens were then compared to that of a flat specimen. The article concludes that shear strength for an arched specimen is reduced (relative to a flat specimen) as the radius of curvature becomes smaller. It is this conclusion that the submitter uses to challenge the validity of the DPO panel report's description of the beneficial effects of wall curvature.

Staff review of the ACI article finds that while it provides useful engineering insights, the conclusions are not directly applicable to the behavior of the SB subjected to aircraft impact. The experimental results cited in the article are based on test specimens with no shear reinforcement and use rollers/hinges to preclude arching action (or membrane behavior). As such, the results are applicable to arch structures governed by beam shear rather than membrane behavior.

[

] [a,c] Further, the test results cited in the email relate to a structure without shear reinforcement. The SB has shear reinforcement in the form of shear ties which resist shear failure modes. The detailed finite-element analyses (FEAs), which were capable of predicting shear failure modes, have provided staff with insights on the controlling failure modes of the SB. These various analyses have shown that out-of-plane displacements of the SB shell are large and that the shell structure resists impact loads with a combination of shear, flexure, and membrane behavior. The modeling results also show that membrane behavior, excluded in the aforementioned experiments, plays a primary role in increasing the SB resistance to aircraft impact.

Based on the above, staff finds that the ACI article cited by the submitter does not invalidate the DPO panel's conclusions regarding the effects of curvature and does not alter the staff's conclusions regarding the acceptability of the SB design.

DPO Panel's Supplementary Evaluation of the Submitter's May 28, 2014 Email on Curvature Effects

On page 33 of the DPO Panel Report the Panel states,

> ***"The fundamental difference between the three thicker walls that were all perforated and the thinner AP1000 shield building SC wall that was not perforated is the fact that all the walls that were perforated are flat plates, whereas the shield building SC wall that was not perforated is a curved cylindrical shell."***

In the May 28th email the Submitter states,

> ***"A paper in the most recent American Concrete Institute (ACI) Structural Journal/May-June 2014 contradicts the curvature effect on shear strength of the AP1000 shield building wall as claimed in the DPO Panel's report."***

The important point, which seems to have been overlooked by the Submitter in reviewing the ACI paper, is that the test specimen results reported in the paper are for curved beams, which bear no resemblance to arches or cylindrical shells like the SB wall.

Throughout the ACI paper cited by the Submitter, the paper's authors refer to the tested specimens as "arch-shaped members," "beams with varying curvature" or "simply supported beams." All of the free-body diagrams presented in the paper (Figures 3(a), (c) and (e), and Figure 5(a)) show only moment and shear forces, and no axial compressive forces acting on the free-body. In addition, the boundary conditions for the test specimens do not allow axial compressive forces to be developed, as would be necessary for the specimen to function as an arch or cylindrical shell. In describing the test setup on page 577, the authors state

> ***"Two bearing plates... were placed over the support rollers. They allowed for free horizontal movements and rotations."***

These boundary conditions do not allow any compressive axial forces to be developed. The tested specimens, therefore, are neither arches nor shells and bear no resemblance to the SC panel walls of the AP1000 SB. Curvature by itself does not make a beam into an arch or shell; it only makes a curved beam. What is required to make an arch or shell is curvature plus lateral restraint so that axial compressive forces can be developed.

The Submitter has completely misconstrued what was reported in the ACI paper in thinking that the results are applicable to the SC panel cylindrical shell wall of the SB. The results in the ACI paper are <u>not</u> applicable to the SC panel cylindrical shell wall at all.

Enclosure 7 – DPO Panel's Assessment of the Submitter's Email Comments of March 27, 2014

7.1 - The Panel's Assessment of the Submitter's Email of March 27, 2014 Titled: "Review of Differing Professional Opinion (DPO) Panel Report Regarding Structural Integrity Concerns with the AP1000 Shield Building (DPO-2012-002)"

Comment 1:

On page 21 of the email, the Submitter states the following:

> ***3.1 The DPO Panel had used false and misleading information.***
>
> ***The Panel's report states that "All four of the consultants engaged by RES also agreed that the demand-to-capacity ratio was acceptable with sufficient margin." This statement is false and misleading.***

The Panel's statement that "All four of the consultants engaged by RES also agreed that the demand-to-capacity ratio was acceptable with sufficient margin" is true and not misleading. Table 1 - "Summary of Advice from RES's Outside Experts" found on page 38 of Reference 1 is a summary of input received from the four RES outside experts related to the issue of the ductility of the shield building wall design in the context of the safety of the shield building SC wall to resist earthquake loads. Item 3 of Table 1 requests the experts advice with respect to the statement "Demand to capacity ratio acceptable with sufficient margin." All four experts, including Dr. Vecchio, responded "Yes." Therefore, the Submitter's statement that "The DPO Panel had used false and misleading information" is completely wrong, and not consistent with the facts documented in Reference 1.

Comment 2:

Throughout the email the Submitter continuously refers to the SC panel wall as "brittle." For the design basis loads that the shield building has been designed to resist, the SC panel wall will not behave in a brittle manner, because of the way the Purdue test results were used to execute the design.

Apparently, the Submitter has failed to recognize that the failure mode of both an RC beam and SC panel changes from ductile to brittle as a concentrated load moves closer and closer to a support. This is what has been observed in dozens of RC beam tests and is exactly what was observed in the three SC panel tests conducted at Purdue University.

The Figure below illustrates a typical brittle shear-compression failure of a RC beam. Reference 2 on page 91 states that "A shear-compression failure can be expected to occur when the shear span a, as indicated in the Figure, is less than <u>four</u> times the beam depth..." The SC panel test that the Submitter always cites as demonstrating the [

] a,c Based on the above, there is no reason to expect an RC beam to behave any differently from an SC panel as far as shear failure is concerned.

At low shear-span-to-beam-depth ratios both RC beams and SC panels fail in a brittle shear mode. At larger shear-span-to-beam-depth ratios both RC beams and SC panels fail in a ductile flexural mode.

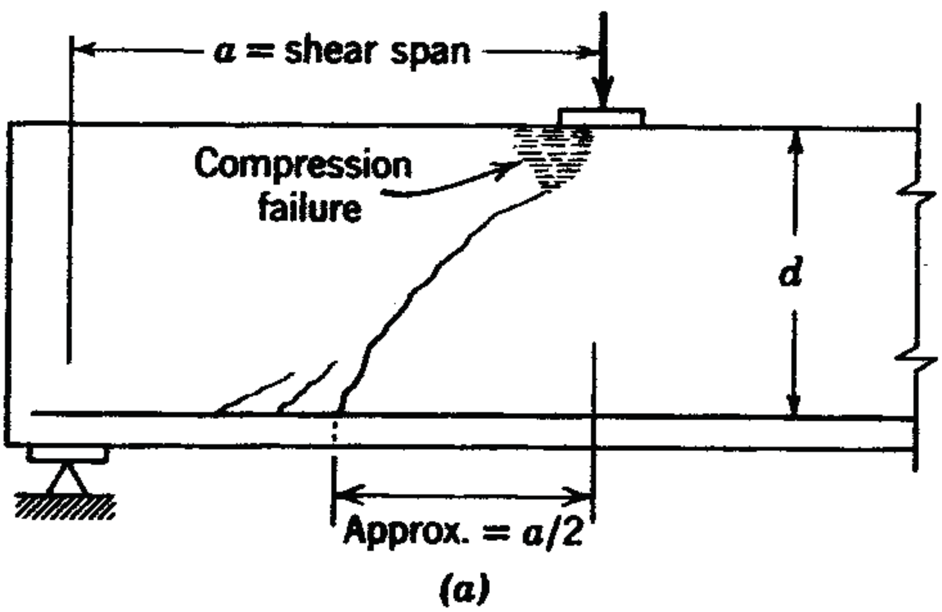

Figure showing a shear-compression failure in a reinforced concrete beam.

In describing the SC panel wall with [ ] a,c tie rod spacing as brittle, the Submitter only ever refers to one of the three SC panel tests. The one test, shown as the red curve in the figure on page 12 of the email (see also DPO Panel Report page 31) is for the test that had a shear-span-to-beam-depth ratio of [ ] a,c For the same SC panel tested with a shear-span-to-beam-depth ratio of [ ] a,c a very ductile failure occurred with a ductility ratio of [ ] a,c However, the Submitter fails to mention this test in any discussions.

To ensure ductile behavior of the SC panel walls everywhere in the SB, WEC used SC panels with an [

] a,c

[ ] a,c ***Therefore, the SC panels in all regions of the SB will behave in a ductile manner in response to design basis seismic loads.***

<u>Comment 3:</u>

On page 22 of the email the Submitter states the following:

> ***4. The DPO Panel's claim that the ACI Code does not require a P-Delta analysis is incorrect.***

The terminology "P-Delta" is a universally understood term in structural engineering. The P-Delta effect refers to the change in overturning moment at the base of a sufficiently tall structure when it is subject to a lateral displacement. The P-Delta effect is a moment equal to the vertical gravity forces multiplied by the horizontal displacement a structure undergoes when loaded laterally by either seismic inertia loads or wind loads.

The ACI-318 and 349 Codes do not mention P-Delta anywhere. Mr. Loring Wyllie, the Panel's expert, has been a continuous member of the ACI 318 Code Committee for over 40 years. Mr. Wyllie stated that "the ACI 349 Code does not specifically require consideration of P-Delta effects..." (See DPO Panel's Report page 59.)

The Submitter cites ACI-318-11, Section 10.10 as a basis for his contention that the ACI Code requires consideration of P-Delta effects. The title of Section 10 is "Slenderness effects in compression members;" it has nothing to do with the P-Delta effects of an overall building structure.

The P-Delta effect is addressed in industry standard ASCE/SEI 43-05. It states that "P-Delta effects shall be included... if the inclusion of P-Delta effects results in greater that a 10% increase in imposed moment demand..." The Panel performed an independent P-Delta calculation on the SB and showed that the increase in the stress resultants at the base of the SB due to the P-Delta effect is less than 1%. (See DPO Panel Report page 59-60.)

On page 23 of the email the Submitter states the following:

> ***I doubt the DPO Panel's calculation on the P-Delta effect had included the reduced wall stiffness resulting from the consideration of concrete cracking.***

In the Panel's original P-Delta calculation concrete cracking was not considered. Considering the effects of concrete cracking by reducing the flexural rigidity of the SB by a factor of 10, the increase is stress resultants at the base of the SB due to P-Delta effects is still less than 1%.

Comment 4:

Regarding the Panel's discussion of the Submitter's figure of the comparison of the SC panel and RC beam tests found on pages 31 and 32 of the Panel's Report, the Submitter states on page 13 of the email,

> ***I had specifically requested that Dr. Hsu double check and verify the correctness and applicability of the diagram...***

On page 13 of the email the Submitter also states:

> ***The Panel's argument [***
>
> ***] a,c significant yielding and ductility would have been observed" is incorrect because the brittle failure occurred due to the fracture of the tie wire, not the yielding of the steel plate.***

In the DPO Report the Panel never disputed the correctness of Dr. Hsu's calculations. What the Panel contends is that the comparison of the SC panel to the RC beam should never have been made because they were each subjected to entirely different loading schemes. In the Purdue tests the SC panel was [ ] a,c whereas the RC beam used in the comparison was loaded by transverse loads and axial compression. It is a classic example of an apples to oranges comparison from which erroneous conclusions are drawn.

The Panel never said that the failure was caused by the yielding of the steel plate. The Panel merely noted that at the time of [ ] a,c and had the SC panel's shear strength been [ ] a,c greater due to a [ ] a,c axial compressive stress (which the RC beam had) significant yielding of the steel plates would have occurred and significant ductility would have been observed in the response of the SC panel. (See DPO Panel Report pages 31 & 32)

Comment 5:

With respect to the aircraft impact assessment of the PCS tank, the Submitter has taken considerable liberty in interpreting statements made by the Panel in the DPO Panel Report. For example, on pages 45 and 46 the DPO Panel Report states, "A simple hand calculation using the impulse-momentum equations shows that the impact may generate sufficient force to create an average shear stress at the base of the outer wall that is in excess of the ACI-349 allowable shear stress of [ ] a,c "The Panel concludes that it cannot provide an independent opinion regarding reasonable assurance of the validity of the WEC assessment of the PCS tank..."

Based on these statements the Submitter on page 5 of the email states "The DPO Panel's report stated that: a) the panel made an independent calculation and concluded that the PCS tank anchorage could be sheared-off." "The DPO Panel... confirmed my safety concern that the PCS tank would fail under aircraft missile impact,... "

It should be clear from the statements in the Panel's Report that the Panel never "<u>confirmed</u>" the Submitters "safety concern that <u>the PCS tank would fail</u> under aircraft missile impact... " (emphasis added)

NOTE: In any simple hand calculation of aircraft impact response, assumptions, mostly conservative, are made to arrive at an estimate of the response. Such is the case here. What the calculation of shear stress tells us is that the effect of aircraft impact on the PCS tank could be very significant and is one of the reasons the Panel recommended that the staff follow-up on the issue.

<u>References:</u>

1. "Summary Evaluation Report: Enhanced Shield Building Structure, Westinghouse Electric Company, Report APP-1200-S3R-003 Revision 2 AP1000, Design Control Document Revision 17," Office of Nuclear Regulatory Research, U.S. NRC, October 29, 2010.

2. "Reinforced Concrete Fundamentals," P.M. Ferguson, John Wiley & Sons, Inc., Third Edition, 1973.

Enclosure 8 – DPO Panel's High Level Summary of September 8, 2014 Meeting with the Submitter

On September 8, 2014, the Panel met with the submitter regarding whether he had any additional information to present to the Panel, as the Panel was completing the Supplemental review. Present at the meeting in addition to the submitter were:

Panel members: [redacted] and Hackett

Joe Williams, NRO

Jim Steckel, NRO

Glenn Tracy, NRO

David Solorio, OE

The discussion focused on key points of difference in opinion between the submitter and the Panel. It is the Panel's judgment that no new information was presented for consideration. Rather the discussion highlighted differences of opinion which were known from prior email and other communications. In an email dated September 9, 2014, the submitter documented three key areas of difference with the Panel: (1) Licensing basis for the design – degree to which the AP1000 shield building was designed in accordance with ACI-349; (2) Design via analysis versus design to a Code; and (3) Adequacy of the shield building wall with the larger tie bar spacing. The Panel concurs that these are areas where fundamental disagreement remains.

These and other differences have been highlighted and addressed previously in Enclosures 5-7.

Enclosure 9 - DPO Panel's Assessment of the Staff's AIA Inspection of the PCS Tank and Containment (Item 4, Enclosure 2)

<u>Introduction and Background:</u>

The Panel was tasked to provide its independent and objective assessment of the technical issue (c) identified in the submitter's Safety Concern #2 that concerns with the structural integrity of the PCS tank and steel containment to withstand physical and shock effects due to aircraft impact:

> ***(c) The connection between the shield building roof and the water storage tank is not adequate. Failure of the connection may lead to an impact of the water storage tank onto the containment and the potential compromise of the primary system. (Items 10 and 12 from the SOC attachment)***

The DPO Panel, due to insufficient resources, could not perform its own independent and objective assessment of the PCS tank and the steel containment for the aircraft impact and shock loads. Therefore, the Panel for its independent and objective evaluation relied on: (1) interviews with WEC analysts, the performer of the AIA of these structures, and the NRC staff that conducted the AIA inspection as required by the AIA regulations; (2) independent and objective review of the regulatory processes in-place (i.e., staff generated inspection reports of the AIA assessment, and the ACRS letter report); and (3) independent and objective review of other documentation.

The Panel was told by WEC (Section 4.2.4 of the DPO report) that the NRC staff did not inspect the WEC assessment of the structural integrity of the PCS tank and steel containment to withstand physical and shock effects due to aircraft impact. The Panel recognized that the ACRS performed an overview of the assessment but the Committee did not conduct a quantitative verification of the WEC analyses but recommended that the staff determine the need for further inspections.

In our DPO Report, we concluded that that we could not provide an independent opinion on the reasonable assurance of the validity of the WEC assessment of the PCS tank impact and the steel containment assessment for potential debris impacts. Therefore, the Panel recommended that the staff follow-up on this issue and perform a review of the WEC assessments of these critical structures.

<u>Supplementary Information</u>

In response to Panel's recommendation for follow-up, the staff provided supplementary information and discussed its response during the May 19 and June 5 , 2014 meetings. The staff provided a written response to the DPO 2012-002 Panel report on July 24, 2014. The Panel was tasked to supplement its report to document its views on the staff's supplementary information.

Issue:

In our DPO Panel Report, we could not establish an objective and independent basis to conclude that the Staff had inspected the Westinghouse Electric Corporation (WEC) detailed structural assessment of the PCS tank and the steel containment for aircraft impact and shock loads.

The staff responded (in the NRO report of July 24, 2014 and augmented the response during the meetings with the Panel) that they had looked into this issue during a series of inspections, between May and August 2011, and confirmed that they have reasonable assurance that under an aircraft impact scenario on the PCS tank, that the PCS tank will not be penetrated and that the containment will remain intact.

Panel's Review of Supplementary Information

The Panel reviewed the staff report 'RESPONSE TO THE REPORT OF THE AD HOC PANEL FOR DPO 2012-002', dated July 24, 2014 and specific references (ADAMS Accession No. ML112650748 and ML111810034). The Panel also reviewed the DPO report for the information related to Panel's interviews with WEC.

1. ***In interviews with WEC, the Panel was informed that:***

    - ***"NRC Staff did not review the PCS tank aircraft impact assessment." (DPO Panel report, Page 29)***

    - ***"NRC Staff had not reviewed the steel containment assessment for the potential falling debris." (DPO Panel report, Page 30)***

2. ***Staff on page 12 of its report 'RESPONSE TO THE REPORT OF THE AD HOC PANEL FOR DPO 2012-002', dated July 24, 2014, provided a reference to ML112650748 to support its basis that the Staff performed inspections of the assessment of the PCS tank and the Steel Containment for the aircraft impact and shock loads.***

    > ***"Subsequent to the January 2011 ACRS letter, the staff performed several additional AIA inspections (May 2011, June 2011, and August 2011). These inspections focused on a number of AIA-related areas, including the WEC analysis of the PCS tank and shield plate, and are documented in an October 2011 inspection report (ADAMS Accession No. ML112650748)"***

    Based on its review of the inspection report, the Panel finds that the report did not support the staff's assertion that they inspected the PCS tank and the steel containment structural assessment for the aircraft impact and shock loads.

The Panel understands from the Inspection Report Number 05200010/2011-202 that:

> ***"The NRC inspection team performed a general review of shield building calculations relating to aircraft impact. As part of this review, the inspection team identified an open item in a calculation relating to the aircraft impact structural analyses that raised questions relative to the application of the LS-DYNA computer code (the computer code used to model the applicable structures and the effects of an aircraft impact on those structures). As a result, the NRC inspection team focused part of this inspection on the computer modeling activities performed in support of the aircraft impact assessment (AIA). In addition, the NRC inspection team reviewed the quality assurance corrective action process and activities for the violation cited in the AP1000 AIA inspection report dated October 28, 2010."***

The Panel's understanding of the scope of the NRC inspection of the three elements in the vendor inspection report is as follows:

2.1. Shield Building Calculations - Inspection Scope

> ***"Specifically, the NRC inspection team reviewed the following shield building structural calculations associated with the design of the shield building cylindrical wall; the design of the conical roof region including steel roof beams; steel concrete (SC) composite to reinforced concrete (RC) mechanical connections; and the connection of the auxiliary building roof with the shield building SC wall for this inspection area."***

The Panel notes from the list of specific shield building structural calculations that none of the calculation titles contain "PCS Tank" or "steel containment".

2.2. LS-DYNA Code –Inspection Scope

> ***"The NRC inspection team reviewed the LS-DYNA modeling activities related to the AP1000 aircraft impact assessment (AIA) structural analysis to verify compliance with the requirements of 10 CFR 50.150, "Aircraft Impact Assessment." This review included an evaluation of the corrective actions implemented by WEC in response to Issue Report (IR) 10-307-M008, "Severity Level IV Notice of Violation Issued to Westinghouse on October 28th," dated November 3, 2010. IR10-307-M008 was written in response to a WEC peer review to ensure that the LS-DYNA code modeling activities were effectively implemented and reflected a realistic analysis in determining whether the shield building as a design feature is functionally capable of withstanding an aircraft impact. As part of this inspection activity, the NRC inspection team evaluated simplified calculations used by WEC in its LS-DYNA code modeling activities to verify that the calculations used accurately reflected the "calculations of record" and the reported results of the AP1000 AIA."***

The Panel notes that the inspection scope was developed primarily to assure the staff that the LS-DYNA code modeling techniques used in the aircraft impact assessment (AIA) including the mesh densities and the calculated solution time steps were appropriate and sufficient to achieve solution convergence. The Panel further notes that the inspection scope did not identify PCS tank or the steel containment structures.

2.3. Corrective Action Program –Inspection scope

> *"The NRC inspectors reviewed WEC's policies and procedures that govern the corrective action process to ensure that they adequately describe the process and implement the requirements of Criterion XVI, "Corrective Actions," of Appendix B to 10 CFR Part 50. The NRC inspection team reviewed WEC's corrective action program and activities, including documentation and records, and discussed the corrective action process with responsible WEC management and staff."*

The Panel notes that the inspection scope did not identify the PCS tank or the steel containment structures. A word search of the inspection report did not reveal words like, "PCS Tank" or "steel containment".

3. *Staff on page 20 of the report 'RESPONSE TO THE REPORT OF THE AD HOC PANEL FOR DPO 2012-002', dated July 24, 2014, provided reference to ML112000530 to support its basis that the Staff performed an <u>audit</u> of the design of the Shield Building PCS tank in June 2011. Later, on August 8, 2014, the staff corrected this reference to ML111810034.*

> *"The staff performed an audit of the design of the SB PCS tank in June 2011 (ADAMS Accession No. ML111810034) and found that the design approach considered appropriate hydrodynamic pressures (convective and impulsive) caused by SSE demands (horizontal and vertical) and were consistent with those cited in the DCD."*

Based on the review of the Staff's design audit report (ADAMS Accession No. ML111810034) of the Shield Building PCS tank, the Panel finds that the report provides WEC responses to the staff's audit questions regarding the PCS tank design for design basis seismic and thermal loads. However, the report does not address the inspection of the PCS tank or the steel containment structural assessment for the aircraft impact and shock loads. A word search of the audit report did not reveal words like, "Aircraft", "impact" "AIA" or "steel containment" or any combination thereof.

4. *Interactions with Staff ([redacted]*

The Panel had an opportunity to interact with [redacted] to better understand whether or not, or to what degree the staff performed the AIA inspection of the PCS tank and the containment for the certification of the AP1000. [redacted] was the Chief of the Structural Engineering Branch responsible for the AIA inspections

of the shield building and containment structures. The interactions consisted of e-mails and a teleconference call on October 9, 2014. The Panel had provided him with a list of specific technical questions related to the PCS tank and the steel containment. The Panel understands that, being the Branch Chief, he did not perform the detailed technical review himself, but rather oversaw the staff efforts in this regard. As such, he was not able to directly answer all of the Panel's detailed questions.

[redacted] ***reiterated the staff's high-level conclusions in the NRO Report of July 24, that the staff inspections of AIA for the AP1000 included the PCS tank impact scenario and the potential impact on the containment. He stated that in the inspection report of October, 2011 (ADAMS ML 1126507), certain aspects of the AIA technical inspection, including the PCS tank scenario, were documented at a higher level since no issues were identified by the staff during the inspection in these areas.***

DPO Panel's Conclusion:

The Panel understands that the principal finding on this issue from WEC and the staff is that the PCS tank is not penetrated by the aircraft impact. In subsequent analyses conducted by WEC in response to ACRS questions, WEC maintained that even if structural failure of the PCS tank were to occur, the debris generated would be insignificant and would not cause failure of either the radiation shield plate or the containment. The ACRS was satisfied with the WEC response on this issue and the staff stated that they conducted further inspections of the WEC assessments to provide added assurance. Following further deliberation on this issue, the Panel finds no reason to doubt the assessments made by the staff, WEC and ACRS on this issue.

However, we do note that we were not able to perform our own independent assessment of this issue due to the lack of availability of certain portions of the documentation (Safeguards information maintained at the applicant's site) and general resource limitations. Therefore, the Panel relied on: (1) interviews with WEC and NRC staff, and (2) appropriate documentation (inspection reports and the ACRS letter on the subject). As noted in Enclosure 9 above, the inspection reports that the Panel reviewed did not contain documentation at a specific enough level to confirm details of the staff's inspection of the PCS tank impact assessment and any postulated debris impact on the containment.

Enclosure 10 – DPO Panel's Assessment of Supplementary Information Regarding the Staff's Seismic Evaluation in FSER Section 3.7 (Item 14, Enclosure 2)

Issue:

The DPO Panel could not confirm the staff's basis for not performing and documenting the evaluation of certain seismic sections of the DCD in the SER supplement for the certified AP1000 design.

The Panel in its report had recommended that the staff follow up on this issue to determine whether the seismic analyses of the certified AP1000 design have been evaluated and documented in the FSER for the Standard Review Plan (SRP) acceptance criteria 3.7.2.II.9 through 3.7.2.II.14.

In response, the staff in its report, 'RESPONSE TO THE REPORT OF THE AD HOC PANEL FOR DPO 2012-002', dated July 24, 2014, on page 27 stated that:

> ***"The staff's evaluation of each listed item is described in the Final Safety Evaluation Report Related to Certification of the AP1000 Standard Design (NUREG-1793, Initial Report; September 2004, ADAMS Accession No. ML060330557). Because the amendment to Revision 19 of the AP1000 DCD did not make changes/revisions to these application areas, staff did not perform a re-review in accordance with design finality. The applicant did not change these sections of the application and, therefore, the staff's conclusions on these sections did not change and were not reiterated in the SER supplement.(emphasis added)***

Information Reviewed

The Panel notes that it could not access the NUREG-1793, Initial Report; September 2004 via the staff referenced ADAMS Accession No. ML060330557. Rather, we accessed it via the following URL: http://www.nrc.gov/reading-rm/doc-collections/nuregs/staff/sr1793/.

The Panel focused its review on verifying the staff's basis as to whether the applicant made changes to certain seismic sections in Revision 19 of the AP1000 DCD.

The Panel noted from its review that the staff provided its evaluation of the following four DCD subsections although not under the corresponding subsections in the FSER.

> ***3.7.2.9 Effects of Parameter Variations on Floor Response Spectra***
> ***Staff provided its evaluation in FSER Subsection 3.7.2.5***
>
> ***3.7.2.10 Use of Constant Vertical Static Factors***
> ***DCD stated that constant vertical static factors are not used for the design of seismic Category I structures.***

3.7.2.13 Determination of Seismic Category I Structure Overturning Moments
Staff provided its evaluation in FSER Subsection 3.8.5.1.3

3.7.2.14 Analysis Procedure for Damping
Staff provided its evaluation in FSER Subsection 3.7.1.2

However, the Panel did not find staff's evaluation for the following two subsections, although the applicant has made changes to these two DCD Subsections, 3.7.2-11 and 3.7.2.12.

A. Section 3.7.2. 11 Comparison of Responses

The staff's review scope under this section is to (i) review the method to consider torsional effects in the seismic analysis of seismic Category I structures (ii) evaluate conservatism of any approximate methods to account for torsional affects in the seismic analysis and design and (iii) review the consideration of accidental torsion for calculating structural responses.

The staff on page 3-89 of the initial FSER (Section 3.7.2.11 NUREG-1793, Initial Report; September 2004, ADAMS Accession No. ML060330557) concluded that the applicant has adequately included the eccentricities due to the mass and member stiffness, as well as the accidental eccentricities,

"3.7.2.11 Method Used to Account for Torsional Effects

Seismic responses of structures, such as in-plane shear in structural elements and in-structure response spectra, are typically affected by torsional effects due to eccentricities between the center of mass and center of rigidity of the structure. Based on its review of DCD Tier 2, Section 3.7.2.3, and its review of the design calculations during the November 13–15, 2002, audit, the staff finds that known eccentricities were explicitly represented in the NI lumped-mass stick model used for the seismic analyses. Also, eccentricities in the steel containment vessel that are associated with the two equipment hatches, the two personnel airlocks, and the polar crane trolley (which is to be parked at one end of the polar crane near the containment shell, as described in DCD Tier 2, Section 3.7.2.3.2) were also explicitly included in the lumped-mass stick model of the steel containment vessel.

From its review of DCD Tier 2, Section 3.7.2.11, "Method Used to Account for Torsional Effects," the staff requested that the applicant provide a clear description of the analysis procedures used to determine how the seismic loads obtained from the time-history seismic analysis of the NI stick models were applied to the equivalent static analysis of the FE models for calculating the seismic member forces to be used in the design. In its response to RAI 230.007 and the revised DCD, the applicant provided the analysis procedure and stated that in each given horizontal direction and at each given floor elevation, the maximum seismic floor acceleration and a torsional moment were applied to the FE models of the NI structures for performing the static analyses. The torsional moment applied at a given floor elevation is equal to the product of the maximum floor acceleration, the corresponding lumped mass in the stick model, and the eccentricity (equal to 10 percent of the maximum building dimensions.) One half of the applied torsional moment is treated to account for the effect of accidental torsion.

The other half supplements the seismic torsion effect produced by the applied floor acceleration on the FE model, such that the total seismic torsion acting on the FE model matches or exceeds the seismic torsional moment in the corresponding member of the NI stick model, as determined from the seismic analysis. The structural element forces and moments due to the three components of ground motion are then combined by the SRSS technique or the 100 percent, 40 percent, 40 percent rule. As discussed in Section 3.7.2.6 of this report, the use of the SRSS technique or the 100 percent, 40 percent, 40 percent rule to combine seismic responses due to the three components of ground motion is acceptable to the staff.

On the basis discussed above, the staff concludes that the applicant has adequately included the eccentricities due to the mass and member stiffness, as well as the accidental eccentricities, in the NI lumped-mass stick model used for the seismic time-history analyses of the NI structures."

Subsequently, in Section 3.7.2.11 (Revision 19 of the AP1000 DCD), the applicant made changes to the method used to consider torsional effects in the seismic analysis of seismic Category I structures.

"The seismic analysis models of the nuclear island incorporate the mass and stiffness eccentricities of the seismic Category I structures and the torsional degrees of freedom. For the response spectrum analysis of the nuclear island, the seismic loads are combined by means of the square root of the sum of the squares (SRSS). The equation for SRSS is shown below.

$$[(\alpha A_{NS})^2 + (\alpha A_{EW})^2 + (A_{VT})^2]^{1/2}$$

where,

A_{NS} = maximum element forces due to safe shutdown earthquake (SSE) response analysis in X (north-south)
A_{EW} = maximum element forces due to SSE response analysis in Y (east-west)
A_{VT} = maximum element forces due to SSE response analysis in Z (vertical)
α = factor to account for accidental torsion effect in NS or EW (1.05) "

DPO Panel's conclusions:

- *The staff initially evaluated (in its initial FSER for DCD (Rev. 15)) the method to account for torsional and accidental torsion effects to ensure that it results in a conservative design and concluded that "the applicant has adequately included the eccentricities due to the mass and member stiffness, as well as the accidental eccentricities, in the NI lumped-mass stick model used for the seismic time-history analyses of the NI structures."*
- *Subsequently, the applicant made changes to the method to account for torsional and accidental torsion effects in Revision 19 of the AP1000 DCD.*

- ***Staff's did not review applicant's justification for the use of the revised method to account for torsional and accidental torsion effects to ensure that it results in a conservative design.***

B. SRP Section 3.7.2..12 Comparison of Responses

Staff's Review scope:

The staff's scope under this section is to review a comparison of peak seismic responses for major seismic Category I structures using modal response spectrum and time history approaches if both the time history analysis method and the response spectrum analysis method are used to analyze an SSC. The staff performs this review to judge the accuracy of the analyses conducted.

DPO Panel's Review:

In its FSER of the DCD (Rev. 15), the staff did not perform a comparison of the peak seismic responses on the basis that that the applicant had used the time history analysis method and not the response spectrum analysis method to analyze major seismic Category I structures.

The staff on page 3-90 of the FSER (Section 3.7.2.12 NUREG-1793, Initial Report; September 2004, ADAMS Accession No. ML060330557) stated,

> ***"As stated in DCD Tier 2, Section 3.7.2.1, the applicant used the modal time-history analysis method as the primary method to perform seismic analyses for the NI structures. The response spectrum analysis method was used for the analyses of SC-I components and substructures. Therefore, the applicant deleted this topic from the DCD. As discussed in Sections 3.7.2.1 and 3.7.2.2 of this report, the modal time-history analysis method is an acceptable method for the seismic analyses of the NI structures. On this basis, the staff finds the deletion of this topic from the DCD to be acceptable."***

Subsequently, in Revision 19 of the AP1000 DCD, the applicant made changes to seismic section 3.7.2.1.3, "Response Spectrum Analysis" in Revision 19 of the AP1000 DCD by including the seismic response spectrum analysis as the basis for determining the seismic design loads for auxiliary building and shield building.

> ***Response spectral analysis is used for the evaluation of the nuclear island structures. Response spectrum analyses are used to perform an analysis of a particular structure or portion of structure using the procedures described in Appendix 3G.4.3.1 and subsections 3.7.2.6, 3.7.2.7, and 3.7.3. Seismic response spectrum analysis of the auxiliary building, shield building, and containment internal structure is performed to develop the seismic design loads for these buildings, and the loads generated include the amplified load due to flexibility and the distribution of this load to the surrounding structures.***

DPO Panel's conclusion:

1. ***Staff did not review and evaluate this Subsection on the basis that the modal time-history analysis method was the primary method to perform seismic analyses for the NI structures.***
2. ***The applicant made changes to this seismic section 3.7.2.1.3, "Response Spectrum Analysis" in Revision 19 of the AP1000 DCD to include the seismic response spectrum analysis as the basis for the auxiliary building and shield building seismic design loads.***
3. ***Both the time history analysis method and the response spectrum analysis method are used to analyze the auxiliary building and shield building. Staff' did not include a section 3.7.2.12 in the FSER that provides a comparison of peak seismic responses for the auxiliary building and shield building Category I structures using modal response spectrum and time history approaches to judge the accuracy of the analyses conducted.***

Based on its objective review, the Panel concludes that:

- ***Staff's did not review and evaluate applicant's justification for the use of the revised method to account for torsional and accidental torsion effects to ensure that it results in a conservative design. (SRP Subsection 3.7.2.11)***
- ***Staff's review did not include a comparison of peak seismic responses for the auxiliary building and shield building Category I structures using modal response spectrum and time history approaches to judge the accuracy of the analyses conducted. (SRP Subsection 3.7.2.12)***
- ***From a very cursory review of the FSER, the Panel could not determine whether the staff included the evaluation the two SRP subsections under different FSER subsections.***

DPO Panel's Conclusion:

The DPO Panel could not confirm the staff's basis (i.e., The applicant did not change these sections of the application and, therefore, the staff's conclusions on these sections did not change and were not reiterated in the SER supplement) for not performing and documenting the seismic evaluation of DCD sections 3.7.2.11 and 3.7.2.12 in the SER supplement for certified AP1000 design.

Document 6: DPO Decision

February 27, 2015

MEMORANDUM TO: John S. Ma, Senior Structural Engineer
Division of Engineering
Office of New Reactors

FROM: Glenn M. Tracy, Director ***/RA/***
Office of New Reactors

SUBJECT: DECISION REGARDING DIFFERING PROFESSIONAL OPINION INVOLVING SAFETY OF THE AP1000 SHIELD BUILDING (DPO-2012-002)

1.0 INTRODUCTION

This memorandum documents my decision regarding a differing professional opinion (DPO) you submitted on July 6, 2012,[1] in which you expressed a view that the certified AP1000 shield building design is unsafe. As described below, considerable staff and contract resources thoroughly assessed your concerns. Based on my review of reports describing this effort, I agree with the conclusions of the ad hoc DPO review Panel that the existing design meets the U.S. Nuclear Regulatory Commission (NRC) requirements, and so provides reasonable assurance of public health and safety. Follow-up actions have been identified which, when completed, will improve the clarity and thoroughness of documentation supporting my decision. The basis for my conclusion and a description of the follow-up actions is provided below.

A summary of DPO-2012-002 will be included in the Weekly Information Report when the case is closed to advise interested employees of the outcome of the review. You will also receive correspondence regarding the follow-up actions described herein.

While my conclusions do not align with the views provided in your DPO, you should be confident that I am well aware of your work in ensuring the AP1000 meets our regulatory requirements. Your efforts led to significant improvements to the certified shield building design from earlier proposals. You have also provided insights which inform the follow-up actions I am directing. Your personal commitment to the NRC's safety mission is admirable.

2.0 BACKGROUND

Your July 6, 2012, submittal was accepted for review in accordance with the requirements of Management Directive 10.159,[2] and was designated as DPO-2012-002. With your assistance, an ad hoc review Panel was identified and tasked to perform a complete, objective, independent, and impartial review of your submittal.

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[1] "DPO-2012-002, Certified AP1000 Shield Building Unsafe," July 6, 2012, Agencywide Documents Access and Management System (ADAMS) Accession No. ML12199A174.
[2] MD 10.159, "The NRC Differing Professional Opinions Program," May 16, 2004, ADAMS Accession No. ML041770431.

The Panel met with you on October 18 and October 25, 2012, to discuss the Statement of Concerns (SOCs). The SOCs were finalized on November 29, 2012, and are documented in Appendix A of the Panel's report[3] describing its efforts and conclusions. Four specific concerns were identified:

1. The certified design of the AP1000 shield building does not meet the NRC's seismic margin requirements.

2. The NRC staff's conclusion that the aircraft missile would not penetrate the AP1000 shield building wall is not logical.

3. The certified shield building wall does not possess sufficient strength and ductility to resist an earthquake and/or aircraft missile impact loading that is specified by the NRC.

4. The certified shield building design does not meet General Design Criterion (GDC) 2 requirements.

Concurrently, the Panel's initial review led to a conclusion that additional technical support would be required due to the nature and complexity of the issues you raised. In response to a request from the Panel, I authorized expenditure of contract funds to obtain the specific technical expertise, as requested. The Panel used these funds to retain two experts, Mr. Loring Wyllie and Dr. Joe Rashid, to assist their evaluation. As described in Section 4.0 of the Panel's report, these experts have extensive and detailed knowledge and experience that is directly relevant to the thorough consideration of your concerns.

Over the course of 2013, the Panel thoroughly reviewed information you provided, as well as other documentation; Appendix B of the Panel's report provides a list of records and documents reviewed in the course of the Panel's review. Appendix C lists individuals the Panel met during the course of its work, including other members of the NRC staff, personnel familiar with the AP1000 shield building design at the Westinghouse Electric Corporation (WEC), experts at Sandia National Laboratory, and others. In addition, Appendix D provides a list of e-mail correspondence between you and the Panel. The Panel considered this information and discussions as it developed its report.

As the Panel was completing its report, it received correspondence from Mr. Wyllie[4] regarding two issues he identified in the course of his work. The Panel referred that correspondence to me for action. After further detailed discussion with NRC Office of New Reactors (NRO) staff, Mr. Wyllie indicated the issues had been resolved to his satisfaction.[5] A letter documenting this conclusion was sent to Mr. Wyllie on April 14, 2014.[6]

The Panel provided its report on February 24, 2014. The Panel concluded that the AP1000 shield building meets NRC's requirements for seismic margin and is adequately resistant to

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[3] "Differing Professional Opinion Panel Report on DPO-2012-002 Structural Integrity Concerns With the AP1000 Shield Building," February 24, 2014, ADAMS Accession No. ML14057A580.
[4] Letter, Loring A. Wyllie to [redacted], "AP1000 Shield Building [Degenkolb Job Number B313003.00]," December 13, 2013, ADAMS Accession No. ML15054A439.
[5] "Summary of the January 24, 2014, Meeting Regarding the AP1000 Shield Building Concern," February 24, 2014, ADAMS Accession No. ML14055A479.
[6] Letter, Denise McGovern to Loring Wyllie, "Concerns You Raised To the NRC Regarding the AP1000 Shield Building," April 14, 2014, ADAMS Accession No. ML14105A446.

aircraft impact. The Panel qualified some of its conclusions, and identified areas where it recommended follow-up action by the NRC staff.

On May 19 and June 5, 2014, I conducted meetings with the Panel and other NRC staff in order to ensure my accurate understanding of the issues and the recommendations of the Panel, to request clarification of varying interpretations, and to seek feedback from the panel and other staff as I began to formulate my decision. This discussion included consideration of your March 27, 2014 response to the Panel's report[7] and other supplemental information you provided on May 1, 2014[8] and May 27, 2014.[9] As I requested, these meetings were conducted under the observation of the NRC's Differing Views Program staff and NRO's Open and Collaborative Work Environment (OCWE) Champion to ensure issues were thoroughly discussed and all points of view considered in accordance with the agency's procedures and values. A summary of these meetings was issued on September 8, 2014.[10]

Based on my initial review of the Panel's report and on the understanding I gained in the May 19 and June 5, 2014, meetings, I directed staff in the NRO Division of Engineering (DE) to review the Panel report and provide a response to the Panel's observations and follow-up actions. A report documenting this effort was issued by DE on July 24, 2014,[11] with a supplemental August 8, 2014, memorandum[12] correcting one reference within that report.

On August 14, 2014, I tasked the Panel to provide a supplemental report considering your additional input, and the DE staff's response, and the discussions and insights gleaned from the May 19 and June 5, 2014 meetings.[13] In addition, as part of that effort, I coordinated a meeting between you and the Panel on September 8, 2014 to ensure the Panel understood any additional relevant information. The Panel's supplemental report was completed on October 30, 2014,[14] and included consideration of information you provided and insights from the meetings described above. The Panel concluded that the DE report had adequately addressed the follow-up items from the Panel's report, but noted that a few topics might merit additional consideration; those items form the basis of the follow-up actions which I am directing.

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[7] E-mail, John Ma to Glenn Tracy, "Review of the DPO Panel's Report," March 27, 2014, ADAMS Accession No. ML15054A358.
[8] E-mail, John Ma to Glenn Tracy, "RE: DPO 2012-02 Discussion Meetings," May 1, 2014, ADAMS Accession No. ML15054A374.
[9] E-mail, John Ma to Glenn Tracy, "Physical test data and the critical shear crack theory on the curvature effect on the punching shear strength of reinforced concrete members," May 27, 2014, ADAMS Accession No. ML15054A384.
[10] "Summary of May 19, 2014 and June 5, 2014, Meetings To Discuss DPO 2012-002 DPO Panel Report And Office Of New Reactor Staff Follow Up Actions," September 8, 2014, ADAMS Accession No. ML14251A343.
[11] "Response to the Report of the Ad Hoc Panel for DPO 2012-002," July 24, 2014, ADAMS Accession No. ML15030A308.
[12] "Editorial Correction On Memorandum Dated July 24, 2014", August 8, 2014, ADAMS Accession No. ML15030A287.
[13] "Ad Hoc Review Panel – Differing Professional Opinion on the AP1000 Shield Building (DPO-2012-002)," August 14, 2014, ADAMS Accession No. ML14223B223.
[14] "Differing Professional Opinion Panel Supplemental Report on DPO-2012-002 - Structural Integrity Concerns with the AP1000 Shield Building," October 30, 2014, ADAMS Accession No. ML14307A870.

In a December 5, 2014, e-mail,[15] you provided your views regarding the Panel's supplemental report. In a January 5, 2015,[16] e-mail, you also forwarded messages exchanged with a former NRC employee, which you believe provide some support for your views. On January 21, 2015, I contacted the former NRC employee to discuss your perspectives. On January 22, 2015, I conducted a meeting to discuss the content of both of these e-mails with the Panel and supporting staff. This meeting was also conducted under the observation of NRC's Differing Views Program staff and the NRO OCWE Champion to ensure issues were thoroughly discussed and all points of view considered in accordance with the agency's procedures and values. The Panel concurred in the summary of that meeting, which was issued on February 23, 2015.[17]

The diverse activities outlined above represent a comprehensive effort involving substantial agency resources, which I judged to be necessary, given the importance of the shield building to the overall safety of the AP1000 design. The issues you have raised have been thoroughly examined by the Panel and defensible conclusions reached based on sound engineering principles. This comprehensive and objective effort demonstrates the agency's sincere commitment to an open and collaborative work environment, the differing views program, and the NRC's values.

3.0 DECISIONS

This section first addresses a cross-cutting issue which affects many portions of your DPO. It then summarizes the four topics described by the DPO Statement of Concerns, the conclusions and recommendations from the Panel's reports, and a discussion of the basis for my decision on each item.

3.1 CAPABILITY OF STEEL COMPOSITE (SC) PANELS

A central aspect of your position is an assertion that steel composite (SC) panels used in the AP1000 shield building are weak and brittle. You claim that the SC panels do not have sufficient energy absorption capability to withstand postulated seismic events and aircraft impact. As a result, you expressed a view that such events could result in catastrophic failure of the shield building and potentially lead to the uncontrolled release of fission products.

Based on review of information you provided, and the work performed by the Panel, and NRC staff, and the insights from the meetings described above, I agree with the Panel's conclusion that the SC panels, in fact, have sufficient strength and ductility. The basis for this conclusion is described below.

Sections 4.2.3, 4.2.3.1 through 4.2.3.4, and 4.2.3.5.3 of the Panel's report and Enclosure 7 of the Panel's supplemental report describe the basis for its conclusion that the SC panel has sufficient strength and ductility. Your submittals have often included a figure describing a comparison of tests of an SC panel and a conventional reinforced concrete (RC) panel, which you assert supports your point of view. However, the Panel notes that you refer to only one of

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[15] E-mail, John Ma to Glenn Tracy, "My review results of the DPO panel report, dated October 30, 2014," December 5, 2014, ADAMS Accession No. ML15054A395.
[16] E-mail, John Ma to Glenn Tracy, "Information value from free will vs. under pressure," January 5, 2015, ADAMS Accession No. ML15054A404.
[17] "Summary Of January 22, 2015, Meeting Regarding Differing Professional Opinion (DPO) 2012-002," February 23, 2015, ADAMS Accession No. ML15035A095.

three SC panel tests. The Panel describes how the performance of the SC panel in the test you cite is consistent with the performance expected for a similarly loaded RC panel. The Panel's report (page 31) states that:

> In a test comparing the strength and deformation behavior of two beams, the Panel would expect that, to make a valid comparison, the type of loads applied to each beam would be the same. However, this was not the case. In the RC beam-column test (blue curve) a compressive axial stress of [ ] a,c was applied to the cross section of the RC beam which greatly increased the shear strength of the RC beam specimen. In the SC panel test, in which the panel was tested as a one-way flat plate (red curve), no compressive axial stress was applied. Had a compressive stress of [ ] a,c been applied to the cross-section, the concrete shear strength of the SC panel test specimen would have been [ ] a,c This would have had a dramatic effect on the SC panel test results, because when the shear failure occurred in the [ ] a,c Had the SC panel's shear strength been [ ] a,c greater, significant yielding and ductility would have been observed. Under these conditions the red curve (SC panel test) would have been expected to look similar to the blue curve (RC beam test). The fact that the SOC, the SOC attachment, and DPO filing did not mention that the loading conditions were different in the two tests makes the comparison misleading and the conclusions drawn from the comparison not valid.

Page 32 of the Panel's report described its conclusion, as follows:

> The Panel finds the comparison between the SC flat panel test and the RC beam test to be misleading because it was never pointed out that the loading conditions were different in the two tests. This skewed the results by making the SC panel appear to be less ductile compared to the RC beam, when in fact, had the loading condition for the SC panel been the same as that for the RC beam, both tests would have very likely produced similar results. Therefore, conclusions drawn from the comparison cannot be considered valid.

This topic was also addressed in the Panel's supplemental report when it considered information you provided in your March 27, 2014, e-mail providing your response to the Panel's report. Enclosure 7 of the supplemental report states that:

> In describing the SC panel wall with [relatively wide] tie rod spacing as brittle, the Submitter only ever refers to one of the three SC panel tests. The one test, shown as the red curve in the figure on page 12 of the e-mail (see also Panel Report page 31) is for the test that had a shear-span-to-beam-depth ratio of [ ] a,c For the same SC panel tested with a shear-span-to-beam-depth ratio of [ ] a,c a very ductile failure occurred with a ductility ratio of [ ] a,c However, the Submitter fails to mention this test in any discussions.
>
> To ensure ductile behavior of the SC panel walls everywhere in the SB [shield building], WEC used SC panels with [relatively narrow] tie rod spacing within all regions where the highest out-of-plane shear forces exist and the effective shear-span-to-beam-depth ratio is low, such as where the roof of the AB frames into the SB. In those regions where the out-of-plane shear forces are low and the effective shear-span-to-beam-depth ratio is high, SC panels with [relatively wide] tie rod spacing are used. Therefore, the SC panels

in all regions of the SB will behave in a ductile manner in response to design basis seismic loads.

Specific dimensions given in the original text provide proprietary design details, and so are not repeated here.

In the attachment to your December 5, 2014, e-mail, you expressed a view that a ductility ratio of [ ] a,c is required, citing the views of two NRC contractors, Drs. Vecchio and Hsu, as supporting your position. Your position was presented and discussed with the Panel in the January 22, 2015, meeting. Your views on this topic were also addressed in the Panel's original and supplemental reports. The February 23, 2015, meeting summary states:

> … the SC panel with wide tie rod spacing demonstrated ductile behavior with a ductility ratio of [ ] a,c when tested with a shear-span-to-beam-depth ratio of [ ] a,c, and so exceeds the ductility ratio of [ ] a,c that Dr. Hsu considers reasonable as quoted in page 5 of the Submitter's letter. Brittle behavior of SC panels is seen only for somewhat lower shear-span-to-beam-depth ratios, which is also the case for conventional reinforced concrete.

Section 4.1.2.4 of the Panel's report provides a discussion of the strength of the SC panel, quoting the Panel's consultant, Mr. Loring Wyllie, as follows:

> …the in-plane shear capacity of the SC wall should have been calculated considering the [ ] a,c For in-plane shear, the ACI 349 equations are probably not directly applicable as the [ ] a,c provide for more shear capacity than a normal reinforced concrete wall. Using ACI 349, shear capacity would be limited to 8√fc'hd by Chapter 21. WEC used a capacity of [ ] a,c for shear. The true in-plane shear strength is much greater and dependent on [ ] a,c provide bond of the steel plates to the concrete core.

Page 64 of the Panel's supplemental report also describes an inquiry to RES consultants, requesting their views regarding a statement that "Demand to capacity ratio acceptable with sufficient margin." As stated in the Panel's supplemental report, the experts all responded "Yes."

Your December 5, 2014 e-mail provided your views following your review of the Panel's supplemental report. In your message, you re-iterated your position that Drs. Vecchio and Hsu support your point of view. You also stated that the Panel over-estimated the strength and capability of the SC panels used in the shield building walls, and that the Panel did not address design shear strength for the safe shutdown earthquake (SSE) and the review level earthquake (RLE). These assertions and supporting statements you provided were presented and discussed in the January 22, 2015 meeting with the Panel and supporting staff. After careful consideration of your views, the Panel's conclusions did not change from those stated in its original and supplemental reports. The February 23, 2015 meeting summary provides additional detail regarding the specific points raised in your December 5, 2014 e-mail.

Based on my review of information provided by you, the Panel, the NRC staff, and the insights from the meetings described above, I agree with the Panel's conclusion that the SC panels have sufficient strength and ductility. This conclusion informs my assessment of your individual safety concerns, as discussed below.

3.2 DECISION REGARDING SAFETY CONCERN 1 - THE CERTIFIED DESIGN OF THE AP 1000 SHIELD BUILDING DOES NOT MEET THE NRC'S SEISMIC MARGIN REQUIREMENTS

This section documents my agreement with the Panel's conclusion that the AP1000 shield building satisfies the NRC's seismic margin requirements, so there is reasonable assurance that the design protects public health and safety. As described below, I am directing the NRO staff to confirm WEC has properly addressed documentation inconsistencies in its corrective action program.

3.2.1 Summary of Review

As stated in its report, the Panel concluded that the AP1000 shield building meets NRC's seismic margin requirements. The Panel's report summarizes its conclusions as follows:

> Based on the simplified and conservative assumptions for the HCLPF [high confidence of low probability of failure] estimate for the shield building wall in-plane shear force, the Panel concludes that the HCLPF exceeds the review level earthquake (RLE) of 0.5g and therefore, the thickness of concrete in the shield wall is sufficient to resist in-plane shear generated by the review level earthquake. The Panel further concludes that the shield building wall satisfies the NRC seismic margin requirements.

The Panel's report also identified two proposed follow-up actions pertinent to this topic:

> The Panel also concludes that: (1) WEC did not re-evaluate the HCLPF values consistent with the shield building reanalysis results that are reported in Appendix L and (2) the HCLPF values reported in Table 19.55-1 of the DCD [design control document] are at variance with the ones reported in Chapter 11 (pages 16-23) of the shield building design report.
>
> The Panel recommends that the staff follow-up on these issues.

The Panel's assessment of Safety Concern 1 was discussed in the May 19, 2014, meeting, and a summary of that discussion was provided in section 3.1 of the September 8, 2014, meeting summary memorandum cited above. This discussion included consideration of supplemental information provided in your March 27 and May 1, 2014 e-mails.

Following that discussion, I directed the NRO DE staff to review the Panel report and provide an assessment of the two HCLPF issues recommended by the Panel, as described above. The NRO DE staff evaluation of these items is given in its July 24, 2014 response to the Panel's report. The Panel provided its conclusions regarding the NRO DE staff evaluation in their supplemental report.

One staff action item addressed the Panel's finding that WEC did not re-evaluate HCLPF in a manner consistent with shield building reanalysis results given in Appendix L of the Shield Building Report. Based on review of the staff's July 24, 2014 response, the Panel stated on page 9 of its supplemental report that it considers the staff's response to be acceptable and the issue to be resolved.

The other follow-up item addressed the inconsistency of HCLPF values reported in DCD Table 19.55-1 and those reported in Chapter 11 of the Shield Building Design Report. Pages 11-12 of the Panel's supplemental report states that the Panel does not consider this issue to be significant relative to the DPO concerns, and defers to the NRC staff for ensuring implementation of NRC's quality assurance requirements. The July 24, 2014, NRO DE response to the Panel's report provides the staff's perspectives on this topic. To improve the clarity and thoroughness of documentation on this topic, I am directing the NRO staff to confirm that the WEC quality assurance process has adequately addressed discrepancies in seismic margin estimates.

Your December 5, 2014, e-mail provided your views following your review of the Panel's supplemental report. In your message, you took exception to the Panel's views, and repeated your assertion that the AP1000 shield building wall does not meet NRC's requirements given in 10 CFR 50 Appendix A, General Design Criterion 2. You also asserted that the shield building does not meet NRC's seismic margin requirements. These assertions and supporting statements you provided were presented and discussed in my January 22, 2015, meeting with the Panel and supporting staff. After careful consideration of your views, the Panel's conclusions did not change from those stated in its original and supplemental reports. The February 23, 2015 meeting summary provides more detail regarding specific points raised in your December 5, 2014, e-mail.

3.2.2 Conclusion Regarding Safety Concern 2

Based on my review of information provided by you, the Panel, the NRC staff, and the insights from the meetings described above, I agree with the Panel's conclusion that the AP1000 shield building satisfies the NRC's seismic margin requirements, so there is reasonable assurance that the design protects public health and safety. Follow-up actions described in Section 4.0 will include direction to the NRO staff to confirm and document that WEC has properly addressed documentation inconsistencies in its corrective action program.

3.3 DECISION REGARDING SAFETY CONCERN 2 - THE NRC STAFF'S CONCLUSION THAT THE AIRCRAFT MISSILE WOULD NOT PENETRATE THE AP1000 SHIELD BUILDING WALL IS NOT LOGICAL

As discussed below, based on review of information you provided, the work performed by the Panel, the NRC staff, and insights from discussions in the meetings described above, I agree with the Panel's conclusion that the AP1000 shield building satisfies NRC's aircraft impact assessment requirements. I am directing action by the NRO staff to address documentation issues associated with inspection of WEC's analysis of the PCS tank in order to ensure clarity of the documentary record on this topic.

3.3.1 Summary of Review

The Panel's report addressed four technical issues that you identified as part of Safety Concern 2.

> (a) Out-of-plane shear tests showed that the shield building wall behaved in a non-ductile manner and therefore has insufficient punching shear strength and ductility to resist the aircraft missile.
>
> (b) The NRC's conclusion that the aircraft missile would not penetrate through (perforate) the shield building contradicts the aircraft missile impact results for the Economic Simplified Boiling Water Reactor (ESBWR) and the Advanced Boiling Water Reactor (ABWR) designs that were submitted for design certification.
>
> (c) The connection between the shield building roof and the PCS tank is not adequate. Failure of the connection may lead to an impact of the PCS tank onto the containment and the potential compromise of the primary reactor coolant system.
>
> (d) The structural wall may behave in a non-ductile manner under impact loading.

Overall, the Panel concluded that the AP1000 shield building is capable of withstanding an aircraft impact, but identified areas for follow-up action by the staff. After completion of those actions, the Panel's supplemental report assessed that work, as well as perspectives you provided, and found that the NRO DE staff resolution of the issues was acceptable overall, but that an issue pertaining to staff inspection of analysis of the PCS tank may merit further consideration.

3.3.2 Decision Regarding Safety Concern 2 Technical Issues (a), (b), and (d)

The Panel addressed items (a), (b), and (d) collectively, stating their conclusion, as follows:

> The out-of-plane shear tests (Purdue tests) showed that the SC panel has the same failure mode characteristics as RC beams. At low shear-span-to-depth ratios both fail in a non-ductile manner (shear failure); at larger shear-span-to-depth ratios both fail in a ductile manner (flexural failure); and at intermediate shear-span-to-depth ratios there is a transition between ductile and non-ductile failure modes.
>
> In Section 4.2.3.2 [of the original Panel report] the Panel's expert, Dr. Rashid, demonstrated the significant effect that shell curvature has on the ability of the SC panel to resist aircraft impact. It is this curvature, which is the reason why the SC panel of the SB can survive an aircraft impact without perforation while thicker SC flat panel and RC walls are perforated by the aircraft.
>
> Based on the Panel's review of the aircraft impact analysis performed by Dr. Rashid, the Panel concurs with his conclusion that, "the AP1000 SC shield building is competent to stop a large commercial aircraft without perforation of the wall or threatening the integrity of the containment."
>
> With regard to the effect of external cold temperature on the integrity of the shield wall due to an aircraft impact:

> Panel members [redacted] and Hackett consider that the WEC evaluation of this issue was appropriate and in accordance with the NEI AIA guidance regarding use of best-estimate properties for the beyond design basis assessment (Section 4.2.3.5.1(a)) [of the original Panel report]. Panel member [redacted] provided his perspective and technical bases in Section 4.2.3.5.1(b) [of the original Panel report] and he agrees with the Submitter's concern stated as, "the ability of the shield wall to resist aircraft impact under cold weather has not been substantiated and is doubtful." He recommends that the staff should follow-up on this issue.

The Panel's report also noted that it was unable to determine if WEC had met the intent of NEI 07-13 for benchmarking aircraft impact assessment computer models, and that it could not provide an independent opinion regarding the WEC assessment of the effect of an aircraft impact on the passive cooling system (PCS) tank and the effect of possible debris impact on the steel containment in the event of a PCS tank structural failure. This issue is addressed in Section 3.3.2.3 below.

The Panel's assessment of Safety Concern 2 was discussed in the May 19, and June 5, 2014, meetings, along with views you provided in e-mails dated March 27, May 1, and May 28, 2014. The September 8, 2014, meeting summary describes discussions pertaining to Safety Concern 2 technical issues (a), (b), and (d) in Sections 3.2, 3.5.1.8, 3.5.1.9, 3.5.2.2, and 3.5.2.3, and in Enclosure 2 items 2 and 3. Enclosure 3 summarizes discussion of "Conclusions and Recommendations" provided in your March 27, 2014 e-mail, and provides cross-references to the NRO DE staff's July 24, 2014 response to the Panel's report.

In evaluating the NRO DE staff's response to its report, the Panel also considered your views as provided in the e-mail messages cited above, and your presentation and discussions with the Panel in their September 8, 2014 meeting with you.

3.3.2.1 Shield Building Ductility (Technical Issues (a) and (d))

Technical issues (a) and (d) state that the shield building wall may behave in a non-ductile manner. As discussed in Section 3.1 above, I agree with the Panel's conclusion that the SC panels which form the shield building wall have sufficient strength and ductility.

3.3.2.2 Effects of Arching Action and Comparison to Other Designs (Technical Issues (a), (c), and (d))

Section 4.2.3.2 of the Panel's report describes how arching action can improve the performance of a curved shell structure in comparison to structures using flat plates. The Panel concurred with the results of analyses by the Panel's contractor, Dr. Rashid, which demonstrated that the arched AP1000 SC shield building wall is capable of withstanding an aircraft impact, even though buildings with flat SC or conventional reinforced concrete walls of similar or even greater thickness might be penetrated.

Enclosure 5 of the Panel's supplemental report provides the Panel's views regarding the position you expressed in your May 1, 2014 e-mail that the AP1000 shield building wall should be made as thick as the shield building for another reactor design. The Panel stated that it agrees with the NRO DE evaluation of this issue, which states that your position does not consider "significant aspects that influence the behavior of a wall subject to an impact load, such as reinforcement ratio, wall span/radius, and curvature." The staff evaluation also notes that three independent aircraft impact assessment analyses have verified the adequacy of the AP1000 shield building.

Your May 28, 2014 e-mail forwarded a paper submitted to the American Concrete Institute (ACI) which you claimed invalidated the Panel's conclusions regarding the beneficial effects of building curvature or arching action. Enclosure 6 of the Panel's supplemental report documents that the paper you cite describes a configuration which is unlike the AP1000 shield building. The Panel states that "The results in the ACI paper are not applicable to the SC panel cylindrical shell wall at all."

Enclosure 6 of the Panel's supplemental report also noted that punching shear has not been observed under aircraft impact loading in analyses completed by the NRC and WEC. Sections 3.2 and 3.5.1.8 of the September 8, 2014, meeting summary also discusses this topic, concluding that aircraft impact does not result in punching shear for the AP1000 shield building. After careful consideration of your views, the Panel's conclusions did not change from those stated in its original and supplemental reports.

Based on my review of the information provided by you, the Panel, the NRC staff, and insights from the meetings described above, I agree with the Panel's conclusion that other reactor designs are not directly comparable to the AP1000 shield building.

3.3.2.3 Analytical Model Benchmarking

Enclosure 4 of the Panel's supplemental report addressed the NRO DE staff action regarding benchmarking of the WEC aircraft impact assessment models. The supplemental report noted that the Panel had not been aware of the staff's inspection efforts addressing this topic when the Panel's report was issued. The supplemental report also noted that independent aircraft impact analyses performed by the NRC's Office of Nuclear Regulatory Research (RES), and by the Panel's contractor, Dr. Rashid, also concluded the AP1000 shield building can withstand an aircraft impact, providing additional confidence in the capability of the WEC models.

Based on my review of information provided by you, the Panel, the NRC staff, and the insights from the meetings described above, I agree with the Panel's conclusion that models used for the WEC aircraft impact assessment have been adequately benchmarked.

3.3.2.4 Effect of Cold Temperatures

Enclosure 3 of the Panel's supplemental report addressed the NRO DE staff action regarding the effects of cold temperatures on the AP1000 shield building aircraft impact assessment, which was an issue raised by Panel Member [redacted]

Requirements for aircraft impact assessment are found in NRC's regulations in 10 CFR 50.150. That regulation states that a realistic, as opposed to bounding, analysis shall be performed to show that the reactor core remains cooled, or the containment remains intact; and that spent fuel cooling or spent fuel pool integrity is maintained. A discussion of the distinction between realistic versus bounding analyses in this context is provided in Enclosure 3 of the July 24, 2014 NRO DE staff response. This discussion was repeated in Enclosure 3 of the Panel's supplemental report, which provided Panel Member [redacted] supplemental assessment of cold weather effects. Based on his evaluation of the NRO DE response and his own evaluation, he stated the conclusion that “The shield building wall is expected to resist aircraft impact under cold weather in the United States.”

Based on my review of information provided by you, the Panel, the NRC staff, and the insights from the meetings described above, I agree with the Panel’s conclusion that the AP1000 shield building can withstand an aircraft impact during cold weather. I have not identified a need for additional limitations on siting future AP1000 facilities due to this issue.

3.3.3 Decision Regarding Safety Concern 2 Technical Issue (c) Regarding the Adequacy of the Connection between the Shield Building Roof and the PCS Tank

In its report, the Panel stated that:

> The Panel concludes that that it cannot provide an independent opinion on the reasonable assurance of the validity of the WEC assessment of the PCS tank and the steel containment assessment for potential debris impacts. The Panel recommends that the staff should follow-up on the WEC aircraft impact assessment of the PCS tank and evaluation of debris impact onto the steel containment.

The Panel’s report stated that WEC had conducted an aircraft impact analysis that determined the PCS tank was not penetrated. However, the Panel raised a question regarding whether a shear failure could occur, and recommended follow-up action for further review of WEC’s analyses. In your March 27, 2014 response to the Panel’s report and in your May 1, 2014 e-mail, you expressed a view that the Panel concurred with your safety concern in this regard, and provided some additional discussion.

The NRO DE staff views regarding the adequacy of the PCS tank connection were discussed in the May 19 and June 5, 2014 meetings. The September 8, 2014 meeting summary describes that discussion. The staff’s assessment was documented in the July 24, 2014 response to the Panel’s report.

The Panel’s supplemental report addressed various issues associated with the PCS tank. Supplemental report Enclosure 2 items 4 and 17 describe the staff’s assessment and the Panel’s supplementary evaluation, with additional discussion in Enclosure 9. The Panel’s conclusions were summarized, as follows:

> The Panel understands that the principal finding on this issue from WEC and the staff is that the PCS tank is not penetrated by the aircraft impact. In subsequent analyses conducted by WEC in response to ACRS questions, WEC maintained that even if structural failure of the PCS tank were to occur, the debris generated would be insignificant and would not cause failure of either the radiation shield plate or the containment. The ACRS was satisfied with the WEC response on this issue and the staff stated that they conducted further inspections of the WEC assessments to provide added assurance. Following further deliberation on this issue, the Panel finds no reason to doubt the assessments made by the staff, WEC and ACRS on this issue.
>
> However, we do note that we were not able to perform our own independent assessment of this issue due to the lack of availability of certain portions of the documentation (Safeguards information maintained at the applicant’s site) and general resource limitations. Therefore, the Panel relied on: (1) interviews with WEC and NRC staff, and

> (2) appropriate documentation (inspection reports and the ACRS letter on the subject). As noted in Enclosure 9 above, the inspection reports that the Panel reviewed did not contain documentation at a specific enough level to confirm details of the staff's inspection of the PCS tank impact assessment and any postulated debris impact on the containment.

Your December 5, 2014 e-mail raised several issues regarding the PCS tank anchorage. These assertions and supporting statements you provided were presented and discussed in the January 22, 2015 meeting with the Panel and supporting staff. After careful consideration of your views, the Panel's conclusions did not change from those stated in its original and supplemental reports. The February 23, 2015 meeting summary provides more detail regarding the specific points raised in your December 5, 2014 e-mail.

Based on my review of information provided by you, the Panel, the NRC staff, and the insights from the meetings described above, I agree with the Panel's conclusion that the PCS tank is not penetrated by an aircraft impact. Nonetheless, I agree with you and the Panel that documentation of the staff's inspection of this topic can be strengthened. Therefore, I am directing the NRO staff to take action to provide clear documentation via an inspection.

3.3.4 Conclusion Regarding Safety Concern 2

Based on my review of information provided by you, the NRC staff, the Panel, and the insights from the meetings described above, I agree with the Panel's conclusion that the AP1000 shield building satisfies NRC's aircraft impact assessment requirements, and there is reasonable assurance that the design protects public health and safety. I am directing action by the NRO staff to perform an inspection to confirm that WEC is maintaining records of its aircraft impact assessment in accordance with 10 CFR Part 52, Appendix D Section X, with a particular focus on the PCS tank and any postulated debris impact on the containment.

3.4 DECISION REGARDING SAFETY CONCERN 3 - THE CERTIFIED SHIELD BUILDING WALL DOES NOT POSSESS SUFFICIENT STRENGTH AND DUCTILITY TO RESIST AN EARTHQUAKE AND/OR AIRCRAFT MISSILE IMPACT LOADING THAT IS SPECIFIED BY THE NRC

This section documents my agreement with the Panel's determination that the AP1000 shield building meets NRC's seismic and aircraft impact assessment requirements, so there is reasonable assurance that the design protects public health and safety.

3.4.1 Summary of Review

The Panel's report described its conclusion regarding Safety Concern 3 as follows:

> Overall, the Panel concludes that the AP1000 Shield Building possesses sufficient strength and ductility and is designed to resist a review level earthquake that is larger in magnitude than the SSE (Section 4.1.2.4) [of the original Panel report] and/or aircraft impact loading (Section 4.2) [of the original Panel report].

The Panel's assessment of Safety Concern 3 was discussed in the May 19 and June 5, 2014 meetings, and included consideration of perspectives you provided in your March 27, May 1, and May 28, 2014 e-mails. The September 8, 2014 meeting summary describes discussions pertaining to Safety Concern 3 in Sections 3.2, 3.3, 3.5.1.3, 3.5.1.5, 3.5.1.8, 3.5.1.9, 3.5.2.1, 3.5.2.2, 3.5.2.3, 3.5.2.4, 3.5.2.5, and 3.5.2.6, and in Enclosure 2 items 1, 2, 3, and 4. Enclosure 3 summarizes discussion of "Conclusions and Recommendations" provided in your March 27, 2014 e-mail, and provides cross-references to the NRO DE staff's July 24, 2014 response to the Panel's report.

The Panel's supplemental report concluded that the NRO DE staff's resolution of recommendations from the Panel's report to be acceptable, noting that three topics might merit further consideration. These topics were designated as items 2, 4, and 14 in the staff's July 24, 2014 response and in the Panel's supplemental report.

Item 2 pertains to the consistency of HCLPF values, as discussed in section 3.2.1 above. As noted in section 3.2.2, I am directing action by the NRO staff to confirm WEC has properly addressed documentation inconsistencies in its corrective action program.

Item 4 pertains to the NRO DE staff evaluation of the PCS tank aircraft impact assessment, as discussed in section 3.3.3 above. As noted in sections 3.3.3 and 3.3.4, I am directing action by the NRO staff to conduct an additional inspection to ensure there is clear documentation of the staff's inspection of WEC's analysis of this topic, in accordance with the aircraft impact assessment rule, 10 CFR 50.150. To address Item 4, the Panel also recommended follow-up action to document WEC's assessment of any postulated debris impact on the containment. I am directing the NRO staff to address this aspect of Item 4, as well.

Item 14 pertains to documentation of the basis for the staff's conclusions described in section 3.7.2 of its final safety evaluation report (FSER)[18] regarding the seismic evaluation of Standard Review Plan (SRP)[19] sections 3.7.2.II.11, "Methods Used To Account For Torsional Effects" and 3.7.2.II.12, "Comparison of Responses." The July 24, 2014 NRO DE staff response to the Panel's report states that these topics were addressed in NRC's review of DCD Revision 15, and were not re-reviewed in Revision 19, as there were no changes in the application regarding those topics. I am directing action by the NRO staff to clarify its documentation of its review of these topics, including augmenting its documentation in a memorandum, as necessary.

3.4.2 Conclusions Regarding Safety Concern 3

As discussed in section 3.1, I agree with the Panel's conclusion that the SC panels have sufficient strength and ductility. Sections 3.2 and 3.3 document my agreement with the Panel's conclusions that the AP1000 shield building meets NRC's seismic margin requirements and that the building can withstand an aircraft impact, respectively. Therefore, I agree with the Panel's conclusion that the certified AP1000 shield building wall has sufficient strength and ductility to resist an earthquake or an aircraft impact, and so meets NRC's regulatory requirements for those topics.

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[18] NUREG-1793, Supplement 2, "Final Safety Evaluation Report Related to Certification of the AP1000 Standard Plant Design Docket No. 52-006," ADAMS Accession No. ML112061231.
[19] NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition," http://www.nrc.gov/reading-rm/doc-collections/nuregs/staff/sr0800/.

3.5 DECISION REGARDING SAFETY CONCERN 4 - THE CERTIFIED SHIELD BUILDING DESIGN DOES NOT MEET GENERAL DESIGN CRITERION (GDC) 2 REQUIREMENTS.

This section documents my agreement with the Panel's conclusion that the AP1000 shield building satisfies the requirements of 10 CFR 50 Appendix A, General Design Criterion (GDC) 2, so there is reasonable assurance the design protects public health and safety.

3.5.1 Summary of Review

As stated in its report, the Panel concluded that the AP1000 shield building meets the GDC 2 requirements. The Panel's report summarizes its conclusion, as follows:

> The panel concludes that, notwithstanding the results of its recommendation in Section 4.1.4 [of the original Panel report], the certified shield building design meets the GDC-2 requirements. On the basis of its assessment of Safety Concerns # 1 and 3 in [original Panel report] Sections 4.1 and 4.3 respectively, the Panel further concludes that the AP1000 Shield Building is of sufficient strength and ductility to resist a review level earthquake (0.5g) that is larger in magnitude by [ ] [a,c] than the design basis SSE earthquake (Section 4.1.2.4 [of the original Panel report]).

The Panel's assessment of Safety Concern 4 was discussed in the May 19 and June 5, 2014 meetings. The September 8, 2014 meeting summary states that the capability of the AP1000 shield building to withstand seismic events was addressed as part of the discussion of Safety Concerns 1 and 3, so there was not an extensive discussion focused on Safety Concern 4 in that summary.

GDC 2 sets a requirement that nuclear power plants be designed to withstand the effects of natural phenomena, including earthquakes. While this regulation addresses other natural phenomena, such as floods, your concern addresses only the capability of the AP1000 shield building to withstand an earthquake.

3.5.2 Conclusion Regarding Safety Concern 4

The discussion of Safety Concerns 1 and 3 above documents my agreement with the Panel's conclusion that the AP1000 shield building satisfies NRC's seismic margin requirements, and that the building has sufficient strength and ductility to resist an earthquake. Therefore, I also agree with the Panel's conclusion that the AP1000 shield building meets GDC 2 requirements pertaining to earthquakes.

<u>4.0 FOLLOW-UP ACTIONS</u>

I am directing the NRO staff to complete the following actions to improve the clarity and thoroughness of documentation supporting my decision.

- Confirm and document that the WEC quality assurance process has adequately addressed discrepancies in seismic margin estimates. (Panel supplemental report Enclosure 2, Item 2).

- Perform an inspection of WEC's aircraft impact assessment documentation, focusing on certain subject areas, as described above. (Panel supplemental report Enclosure 2, Item 4, and Enclosure 9).

- Clarify documentation of NRC staff review of the AP1000 shield building for SRP sections 3.7.2.II.11 and 3.7.2.II.12, including augmenting its documentation in a memorandum. (Panel supplemental report Enclosure 2, Item 14, and Enclosure 10).

5.0 CONCLUSION

Based on my review of the information described above, I agree with the Panel's conclusions that the existing AP1000 shield building design meets the NRC's seismic and aircraft impact assessment requirements, and so provides reasonable assurance of public health and safety without changing that design.

While I understand that your DPO seeks additional changes to the AP1000 shield building design, your review efforts have already contributed to significant improvements from the design originally proposed. You have also provided insights which inform the follow-up actions I am directing as described above. I, therefore, extend my thanks for your personal commitment to the NRC's safety mission.

Please note that the contents of this memorandum should not be revealed outside the NRC until further review is conducted and any sensitive information is redacted and approved for public disclosure, consistent with the DPO process described in MD 10.159.

Document 7: DPO Appeal Submittal

NRC FORM 690
(11-2002)
NRCMD 10.159

U.S. NUCLEAR REGULATORY COMMISSION

DIFFERING PROFESSIONAL OPINION -- APPEAL

FOR PROCESSING USE ONLY

1. DPO CASE NUMBER: 2012-002

2. DATE APPEAL RECEIVED: March 31, 2015

INSTRUCTIONS: *Prepare this form legibly and submit three copies to the address provided in Block 12 below.*

3. NAME OF SUBMITTER	4. POSITION TITLE			5. GRADE
John S. Ma	Senior Structural Engineer			7
6. OFFICE/DIVISION/BRANCH/SECTION	**7. BUILDING**	**8. MAIL STOP**	**9. SUPERVISOR**	
NRO/DE/SEB2	Building 2	T-8K09M	Jim Xu	

10. DESCRIBE THE DIFFERING PROFESSIONAL OPINION. DESCRIBE THE PRESENT SITUATION, CONDITION, METHOD, ETC., WHICH YOU BELIEVE SHOULD BE CHANGED OR IMPROVED. *(Continue on Page 2 or 3 as necessary.)*

I demonstrated that (1) the certified AP 1000 shield building does not meet the NRC's seismic margin requirement, (2) there is no adequate demonstration that the shield building meet the 10 CFR Part 50, Appendix A, General Design Criterion (GDC) 2 requirement, (3) the NRC's conclusion that the aircraft missile would not penetrate the AP 1000 shield building wall is illogical in my DPO, dated June 29, 2012. None has been resolved.

This DPO demonstrated that the AP1000 shield building wall is brittle and is insufficiently thick (strong) resulting from improper analysis and design methods that had been used by Westinghouse, improperly reviewed by the staff, and improperly resolved by the DPO panel. I am seeking actions to conduct proper analysis and design review and that will result in a ductile and thicker wall that will meet 10 CFR Part 50, Appendix A, General Design Criterion (GDC) 2 requirement, the NRC's seismic margin requirement, and the NRC aircraft missile impact requirement.

11. DESCRIBE YOUR REASONS FOR SUBMITTING AN APPEAL (IN ACCORDANCE WITH THE GUIDANCE PRESENTED IN NRC MANAGEMENT DIRECTIVE 10.159). *(Continue on Page 2 or 3 as necessary.)*

The DPO decision was greatly influenced and informed by the staff and the DPO Panel's flawed information and its lack of understanding of the pertinent technical issues. I am concerned that this decision will result in the unsafe AP1000 designs (including Vogtle and Summer plants currently under construction) and does not provide a reasonable assurance to public health and safety and the environment. It has taken the staff and the DPO Panel almost two and half years to provide their resolution to shield building wall design issues that I raised in my DPO; "brittleness" in shear, shear strength and safety factor for shear, and the (punching) shear local failure mode as a result of aircraft missile strike of the wall, but none has been resolved correctly.

Some evidence indicating the confusion of the DPO panel's understanding on the pertinent technical issues is provided on page 2. More detail information on the incorrect opinions of the DPO panel is in the attached word file.

SIGNATURE OF SUBMITTER	DATE	SIGNATURE OF CO-SUBMITTER *(if any)*	DATE
John S. Ma	3/31/2015		

12. SUBMIT THIS FORM TO:

Differing Professional Opinions Program Manager

Office of:

Mail Stop:

13. ACKNOWLEDGMENT

13. SIGNATURE OF DIFFERING PROFESSIONAL OPINIONS PROGRAM MANAGER (DPOPM)	DATE OF ACKNOWLEDGMENT
Renée Pedersen	4/1/2015

14. DECISION

☐ Appeal sustained

☐ Appeal denied (see attached)

Differing Professional Opinion Closed

DATE

NRC FORM 690 (11-2002)

NRC FORM 690 U.S. NUCLEAR REGULATORY COMMISSION
(11-2002)
NRCMD 10.159

DIFFERING PROFESSIONAL OPINION
(Continued)

CONTINUE ITEM 10 AND/OR ITEM 11 FROM PAGE 1. *(Indicate the block number to which this information applies.)*

Item 11
Shear ductility of the shield building wall

The Panel revealed only now for the first time that it had found a test indicating that the wall element was more ductile, with a [ ]a,c. The DPO panel's understanding of the "ductility" or "brittleness" issue is in a state of confusion, and its incorrect opinion has misled Mr. Tracy into believing that the SC wall element and the wall are "ductile". The DPO panel's method to find shear ductility is wrong and dangerous to safety.

The DPO Panel's understanding of data comparison between SC and RC elements (red and blue curves) is flawed and it continues to accuse my comparison of the two curves as 'misleading'. That accusation is false and only demonstrates its lack of understanding of the ductility issue for design and took no action to resolve the issue. The fact is that the DPO panel has misled Mr. Glenn Tracy into believing that the test wall was actually ductile in shear. The SC wall element and the wall is "brittle", which is not acceptable in concrete structural design in general and should not be accepted for the AP1000 shield building wall in particular.

Shear strength of the shield building wall
The DPO panel's evaluation on the shear strength of the AP1000 shield building relied on Mr. Loring Wyllie's opinion and judgment. Should the NRC rely on Mr. Wyllie's belief that the [ ]a,c and accept that value instead of the ACI Code's limit of 8√fc', which is the design basis of the AP1000 shield building? While the ACI Code provided the limit of average shear strength of 8√fc' for concrete walls in a building with the requirement that each individual wall shall not exceed 10√fc' as the legal limits, and that the safety factor, the ratio of strength to stress, shall be greater than 1.0 as the basis for safety acceptance. [ ]a,c which is less than 1.0, indicating unsafe design. However, the staff and the DPO panel provided no shear strength values and safety factors in shear for its evaluation on the safety of the AP1000 shield building wall, except stating that the wall is safe.
Furthermore, there were modeling errors of the AP1000 shield building. Westinghouse under predicted the design shear demands and stresses in the shield wall. Had those modeling errors been corrected, the actual magnitudes of shear stress in the wall would have been increased and thus further [ ]a,c
Punching shear wall failure due to an aircraft missile strike

For the strike of hurricane generated automobile missile on the shield wall, Westinghouse only examined the punching shear strength of the shield wall and the NRC reviewer only reviewed the punching shear issue in the NRC SER because [ ]a,c
However, for the aircraft missile strike the DPO panel stated that it was the curvature and not the punching shear strength in the AP1000 shield building wall that resists the aircraft missile impact compared with the flat walls or slabs. The DPO panel insisted on the "curvature" effect as its main reason to justify the adequacy of the wall thickness for AP1000 shield building. In the Panel's opinion, the curvature effect, for example, even overrides the reduction of [ ]a,c Can the small amount of curvature increase from the EPR to AP1000 shield building wall really increase the so called "arching action" so much to push the [ ]a,c to overcome the thickness deficiency?

The design of the AP1000 shield building wall is strong in resisting bending, but weak in shear, and therefore the actual local failure mode of the wall, as a result of aircraft missile strike should be (punching) shear instead of bending. However, the analyses results from the computer codes indicated a bending failure mode, suggesting that the shear failure mode (the red curve) had not been, or had been incorrectly, coded into the computer analytical models.

Appeal to EDO
Regarding NRO Director's Decision on
DPO 2012-002

Safety of AP1000 Shield Building Wall

By

John S. Ma

March 31, 2015

Appeal to EDO
John Ma
March 31, 2015

Executive Summary

I am appealing to you the NRO Director Glenn Tracy's decision on my DPO involving safety of AP1000 shield building because of my belief that we all have an obligation and duty to ensure public health and safety. The DPO decision was greatly influenced and informed by the staff and the DPO Panel's flawed information and its lack of understanding of the pertinent technical issues. I am concerned that this decision will result in the unsafe AP1000 designs (including Vogtle and Summer plants currently under construction) and does not provide a reasonable assurance to public health and safety and the environment.

It has taken the staff and the DPO Panel almost two and half years to provide their resolution to shield building wall design issues that I raised in my DPO; "brittleness" in shear, shear strength and safety factor for shear, and the (punching) shear local failure mode as a result of aircraft missile strike of the wall**.**

Shear ductility of the shield building wall

The Panel revealed only now for the first time that it had found a test indicating that the wall element was more ductile, with a ductility ratio of [ ] a,c than it is required, which is [ ] a,c The DPO panel's understanding of the "ductility" or "brittleness" issue is in a state of confusion, and its incorrect opinion has misled Mr. Tracy into believing that the SC wall element and the wall are "ductile". The DPO panel's method to find shear ductility is wrong and dangerous to safety.

The DPO Panel's understanding of data comparison between SC and RC elements (red and blue curves) is flawed and it continues to accuse my comparison of the two curves as 'misleading'. That accusation is false and only demonstrates its lack of understanding of the ductility issue for design and took no action to resolve the issue. The fact is that the DPO panel has misled Mr. Glenn Tracy into believing that the test wall was actually ductile in shear. The SC wall element and the wall is "brittle"**,** which is not acceptable in concrete structural design in general and should not be accepted for the AP1000 shield building wall in particular.

Shear strength of the shield building wall

The DPO panel's evaluation on the shear strength of the AP1000 shield building relied on Mr. Loring Wyllie's opinion and judgment. Should the NRC rely on Mr. Wyllie's belief that the [ ] a,c was conservative, and accept that value instead of the ACI Code's limit of 8√fc', which is the design basis of the AP1000 shield building? While the ACI Code provided the limit of average shear strength of 8√fc' for concrete walls in a building with the requirement that each individual wall shall not exceed 10√fc' as the legal limits, and that the safety factor, the ratio of strength to stress, shall be greater than 1.0 as the basis for safety acceptance. [ ] a,c which is less than 1.0, indicating unsafe design. However, the staff and the DPO panel provided no shear strength values and safety factors in shear for its evaluation on the safety of the AP1000 shield building wall, except stating that the wall is safe.

Furthermore, there were modeling errors of the AP1000 shield building. Westinghouse under predicted the design shear demands and stresses in the shield wall. Had those modeling errors been corrected, the actual magnitudes of shear stress in the wall would have been increased and thus further [ ] a,c

Punching shear wall failure due to an aircraft missile strike

For the strike of hurricane generated automobile missile on the shield wall, Westinghouse only examined the punching shear strength of the shield wall and the NRC reviewer only reviewed the punching shear issue in the NRC SER because the punching shear strength controls the shield building wall thickness.

However, for the aircraft missile strike the DPO panel stated that it was the curvature and not the punching shear strength in the AP1000 shield building wall that resists the aircraft missile impact compared with the flat walls or slabs. The DPO panel insisted on the “curvature” effect as its main reason to justify the adequacy of the wall thickness for AP1000 shield building. In the Panel’s opinion, the curvature effect, for example, even overrides the reduction of punching shear strength in AP1000 by a factor of four due to its one-half of the thickness of the EPR wall thickness. Can the small amount of curvature increase from the EPR to AP1000 shield building wall really increase the so called “arching action” so much to push the punching shear strength of the AP1000 shield wall up to a factor of more than four to overcome the thickness deficiency?

The design of the AP1000 shield building wall is strong in resisting bending, but weak in shear, and therefore the actual local failure mode of the wall, as a result of aircraft missile strike should be (punching) shear instead of [ ] a,c However, the analyses results from the computer codes indicated a [ ] a,c failure mode, suggesting that the shear failure mode (the red curve) had not been, or had been incorrectly, coded into the computer analytical models.

I sincerely hope that for the sake of public health and safety, you will reconsider the decision to continue with the unsafe design and construction of AP1000 plants.

The details of the basis of my appeal are provided in the following.

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I. Introduction

The major disagreements between the DPO panel's reports and my DPO and the subsequent submittals can be summarized as below:

1. While I consider the AP1000 shield building SC wall element and the wall to be **brittle** in shear, and thus not meeting the requirements of ACI Code, which is the design basis of the AP1000 shield building, the DPO panel considered it to be **ductile** with a ductility ratio of [ ] a,c in shear and meeting the required ductility ratio of [ ] a,c that was jointly established by the NRC and Westinghouse
2. While I consider that the in-plane shear strength of the three feet thick AP1000 shield building SC wall, as specified in the ACI Code, has been exceeded by the in-plane shear stress under loading conditions, including the safe shutdown earthquake (SSE) load, and has been significantly exceeded by the in-plane shear stress under loading conditions, including the review level earthquake (RLE) load, the DPO panel concluded that the shield building wall can resist in-plane shear stress generated by the RLE, and that the wall satisfies NRC seismic margin requirements
3. While I believe that the staff's and the DPO panel's conclusion that the aircraft missile cannot penetrate through the SC wall is illogical, the DPO panel believes its conclusion logical

These three major issues will be discussed in the following sections after a presentation on how safety was determined in ancient Rome and its evolution and in modern days.

- How safety was determined in ancient Rome

The safety of a bridge in ancient Rome was tested after its completion by requiring the design engineer to stand under the bridge while chariots drove over the bridge. They know that the bridge will have to resist the load of chariots passing over it, but had no scientific method available to judge whether the designer's "opinion" that his design was safe was true or not. Therefore, by requiring the bridge designer to risk his own life during the acceptance test was not only a way to hold him accountable but also to motivate him toward a safer design.

- How has the safety issue been evolved and determined in modern days

Galileo was the first one to investigate the behavior of a beam subjected to bending in 1638, and since then laboratory testing was performed to obtain "strength" of structural elements and members. Testing equipment has recently advanced to a point that an entire seven-story building had been placed on a testing equipment of a shake table subjected to input of simulated earthquake ground motions, and the roof movements had been recorded and compared with the movements predicted by a structural analysis method, which had been verified to be correct and applicable to concrete structures, during the entire period of simulated earthquake ground motions (see slides 33 and 34 in reference 1). Structural analysis methods, if verified to be correct, can predict the "stress" in different locations in buildings. Modern Structural engineering analysis and design of concrete structures have long been moved away from individual "opinion", as the ancient Rome did, and instead is aimed at quantifying a building's safety by a safety factor, which is the ratio of "strength" to "stress". The safety factor must be greater than 1.0, and should never be less than 1.0. The greater the safety factor, the safer the design of the building.

Scientific test data on "strength" for structural members, or elements, or entire buildings, and the requirements for proper analysis methods to obtain "stress" in buildings are disseminated in building codes. The proper analysis and design methods to achieve proper safety factors for concrete structures are provided in standards, and recommended practices for specific structural members, such as structural (shear) walls, which are also applicable to the shield building wall, are provided in design guides. Based on the analysis and design procedures and acceptance criteria that are stated in codes, standards, and guides, safety factors for buildings including the shield building can be obtained objectively. These objectively obtained safety factors are used to design or review for the safety of buildings in modern days.

To achieve the safety factor for elements (modules, or sections), and the entire building, the design process for concrete structures, including shield buildings, is a trial-and-error procedure, which proceeds as follows:

1. Design all elements (members, modules, sections) used in the building to be ductile
2. Design the entire building not to fail in brittle modes, such as shear or torsional failure modes or compression buckling mode, but to eventually fail in ductile modes, such as flexure (bending) mode, so that the building will possess a great amount of energy absorption and dissipation capability to counter and resist the energy imparted to the building generated by the earthquake ground motions or missile impact
3. Assume a geometry and a thickness of the wall for the building
4. Subject the assumed building to seismic analysis, such as a SSE or RLE condition, and obtain stresses at all locations or elements (sections) in the building
5. Add the stress from the seismic analysis in step 4 to stresses obtained from other loading conditions in accordance with the ACI Code's loading combination equations
6. Calculate the strength of wall elements based on the equations specified in the ACI Code
7. Compute the ratio of the calculated strength in step 6 to stress in step 5
8. If the ratio is greater than 1.0, it demonstrates that the assumed geometry and wall thickness are appropriate. If the ratio is less than 1.0, which means that the strength is less than the design stress, or the safety factor is less than 1.0, that means the building is unsafe, the wall thickness must be increased to increase the wall strength to make the safety factor greater than 1.0 if the geometry of the building remains the same
9. Repeat step 2 through step 8 until the ratio of strength to stress (the safety factor) is greater than 1.0

The above steps are not only typical but also necessary for the design process of concrete buildings by building designers, and for the review process of concrete buildings by building officials (plan reviewers) for conventional buildings and the NRC reviewers for nuclear power plant buildings. These steps in the design or review process were established and practiced by the structural engineering community, and the acceptance criteria were obtained from scientific test results.

By following these steps, it will be shown that the AP1000 shield building wall does not meet the ductility and safety factor requirements, which indicates that the design is incomplete and unsafe.

- How has the DPO panel determined the safety of AP1000 shield building wall

The DPO panel's safety conclusions were based on "opinion", similar to the approach of the ancient Rome bridge designer's "opinion" minus accountability. Some of the opinions expressed by the DPO panel will be described below to show that the DPO panel's approach by using "opinion" is wrong and dangerous to the safety of the AP1000 shield building.

II. Is the AP1000 shield building wall element and the wall brittle or ductile in shear?

II.1 <u>The wall element was tested "brittle", but became "ductile" in the DPO panel's opinion</u>

II.1.1 The red curve

The AP1000 shield building wall element failed in a **brittle** manner during the shear test, as required in step 1 of the design process. The brittle failure can be seen in the red curve below.

a,c

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II.1.2 The newly found test data by the DPO panel

The DPO panel stated in its latest report (see page 2-2 in reference 2) "As discussed above, the SC panel with wide tie rod spacing demonstrated ductile behavior with a ductility ratio of [ ] a,c when tested with a shear-span-to-beam-depth ratio of [ ] a,c and so exceeds the ductility ratio of [ ] a,c that Dr. Hsu considers reasonable as quoted in page 5 of the Submitter's letter."

II.1.3 The newly found test data is a flexure test, not a shear test

The NRC consultant, Dr. Vecchio, stated in his December 3, 2009 letter (reference 3), "Three full-scale out-of-plane shear beam tests are being proposed. The test specimens will be more consistent with actual Shield Building SC wall details than was the case in the previous plan for these tests, and thus this is an improvement. However, it is being proposed that the three tests be done with [ ] a,c It is my fear that, with these span-to-depth ratios, the tests may be inconclusive towards verifying the out-of-plane shear capacity of the wall detail. [ ] a,c moreover, Takeuchi et al. conducted a number of tests at this span-to-depth ratio, so another test under the same condition will not add significantly to the verification process. At the other extreme, [ ] a,c so it will likely add nothing to the verification study either. My strong recommendation is to conduct the tests under conditions which are likely to be shear-critical and which are not well presented in data available from the literature; [ ] a,c are recommended."

If [ ] a,c is almost surely to be governed by flexural failure as stated by Dr. Vecchio, the [ ] a,c would definitely be governed by flexure failure. As stated in Dr. Vecchio's letter, the [ ] a,c are shear test, and the [ ] a,c is flexure test.

II.1.4 How the shear strength and ductility be determined through tests

As stated above in Dr. Vecchio's letter, that it was his fear that the three tests proposed by Westinghouse may be insufficient to find the lowest value of out-of-plane shear strength for the wall. The purpose for testing the three [ ] a,c for the wall element with a specific design (wall thickness of three feet with [ ] a,c which constitutes a specific shear reinforcement index value as stated under Section 1.2, "Structural Engineering Principles for Seismic Design" of my DPO) was in searching for the **minimum** value of shear strength and ductility for the test wall element only if the test results meet the acceptance criteria. If the test results do not meet the acceptance criteria, a new design for the test wall element would be required. Therefore, even test values from other a/d ratios for the same shear reinforcement index showed more strength and ductility, they should not be used to replace the most brittle value or to average them out because the most brittle value governs the failure and should be used for the design or review of the wall.

II.1.5 The DPO panel's method to find shear ductility is wrong and dangerous to safety

The DPO panel's method to choose the highest value among all tests as the wall's shear ductility is wrong and dangerous to safety because engineers should always choose the lower value for the design in order to be conservative and choosing the lower value is also a common scenes approach.

II.1.6 The DPO panel did not resolve the ductility issue in a timely manner

The DPO panel, after two and half years working on the resolution of this fundamental design issue of "brittleness" in shear that I stated in my DPO, dropped a bombshell revelation for the first time that it had found a test indicating that the wall element was more ductile, with a ductility ratio of [ ] a,c than it is required, which is [ ] a,c The DPO panel even blamed on me for not mentioning this test in the past by stating, "However, the submitter fails to mention this test in any discussion." Why should I have to mention a "flexure" test that has no relevance to my previous submittals, which were only related to "shear"? If the DPO panel had a doubt on why I did not mention this test, it could and should have asked me about it, then we would not have had to waste the two and half year time and still stuck in such a terrible situation in arguing about the most fundamental design issue of shear "ductility" for concrete structures in general and the AP1000 shield building in particular.

II.2 The DPO panel's misunderstanding of data comparison, and lack of action to resolve it

The panel's report, dated February 24, 2014, stated that the SC module #2 (the red curve) would have significant yielding and ductility had a [ ] a,c been applied to the specimen as it did to the RC specimen, and called the comparison misleading and the conclusions drawn from the comparison not valid. I responded in my March 27, 2014 report (reference 4) to Panel's statement as follows:

- The panel's argument "When the shear failure occurred in the [ ] a,c Had the SC panel's shear strength been [ ] a,c significant yielding and ductility would have been observed" is incorrect because the brittle failure occurred due to the fracture of the tie wire, not the yielding of the steel plate. The reason of the failure was so brittle is due to the combination of insufficient amount of tie wires and the brittle nature of [ ] a,c being used, which is not allowed by the ACI Code (60 ksi maximum).
- In response to Gordon Bjokman's email request, dated May 3, 2013, 9:38 am, I forwarded the information "RC vs. SC" to him in the attachment of my email, dated May 3, 2013, 10:03 am.
- In that "RC vs. SC" information email, I had specifically requested that Dr. Hsu to double check and verify the **correctness and applicability** of the diagram, which was prepared, and sent to me, by Dr. Mo. As can be seen in the response of the e-mail, it stated that "Professor Hsu reviewed and **confirmed the correctness and applicability** of the diagram."

- Based on my testing experience on RC members subjected to a combination of axial, bending, and shear loads, it is my judgment that had the axial stress, [ ] a,c which is [ ] a,c been removed from the RC specimen, the RC curve would be lowered slightly with an increase of ductility slightly with the area under the curve (energy absorption/dissipation capability) remained about the same.
- I believe that Dr. Mo's curves in the diagram for comparison purpose, with respect to ductility and energy dissipation/absorption capability is reasonable, and Dr. Hsu's statement on the correctness and applicability of the diagram is also reasonable.

Dr. Mo, Dr. Hsu, and I have all personally conducted shear tests, and it is our conclusion that the comparison is valid.

Since my response, as stated above on March 27, 2014, I have not been contacted by any one of the DPO panel members, or NRC staff members, in an effort to resolve their doubts on the validity of the comparison of the two curves.

After eleven months later, I saw on page 5 of Mr. Tracy's letter (reference 5) that the same issue was still unresolved in the mind of the DPO panel, and it continued to accuse my comparison of the two curves in the first diagram misleading. That accusation is false, and the fact is that the lack of the DPO panel's understanding of concrete testing and behaviors of structural members and its inaction to resolve the issue had misled Mr. Glenn Tracy into believing that the test wall was actually ductile.

> "The Panel finds the comparison between the SC [ ] a,c test and the RC beam test to be misleading because it was never pointed out that the loading conditions were different in the two tests. This skewed the results by making the SC panel appear to be less ductile compared to the RC beam, when in fact, had the loading condition for the SC panel been the same as that for the RC beam, both tests would have very likely produced similar results. Therefore, conclusions drawn from the comparison cannot be considered valid."

Since this issue had influenced Mr. Tracy's decision on the ductility of the SC wall, I am going to address and resolve it below.

First, look at the two curves (red and blue) separately or assume that each curve is plotted in its own diagram with the same coordinates. There is no dispute that each curve truly represents the shear strength and ductility of the respective SC or RC elements. Therefore, the "brittle" nature of the SC wall element, represented by the red curve, is truly **brittle** because the element failed as soon as the tie wire reached its yield strain. On the other hand, when the steel in the RC element yielded to a strain four times its yield strain, the applied force was reversed to the opposite direction until the steel strain reached to four times the yield strain, and this completed a cycle, representing the beam or column oscillating in a building during earthquakes. The RC element failed at 12 cycles, which is considered ductile enough to sustain earthquake motions. Whether the comparison between the two curves is reasonable or not does not change the fact that the SC wall element (the red curve) is "brittle" because it failed at [ ] a,c and the RC element is ductile because it failed after 12 cycles at the ductility ratio of 4.

Does the comparison between the two curves reasonable? The answer is yes. This is because that research has determined that the shear strength comparison is related to the shear reinforcement index. For two different RC beams having the [ ] a,c their shear strengths should be close. As described in my DPO that the ordinary reinforced concrete element RC had a shear reinforcement index pFy=0.16 vs. the [ ] a,c the AP1000 test SC element, which are very close to each other, and therefore the comparison is reasonable and meaningful. The fact that the two curves are [ ] a,c is a testimony that the [ ] a,c approach is valid, and that the influence of the 590 psi axial compressive stress on the RC element while none on the SC element had no effect in the [ ] a,c

As I stated in the fourth bullet above that had the 590 psi axial compression force been applied to the SC wall element, the force would increase the shear strength slightly and reduce the ductility slightly and the energy absorption or dissipation capability will remain about the same based on my testing experience. However, the panel's statement seems to indicate that the influence of the 590 psi axial compressive force was so substantial, that had it been applied on the SC wall element, would have made the two curves look very close, which I do not agree and would call the statement misleading. Assume that the DPO panel's opinion were correct that the behavior of the SC wall element would have been looked like the RC element (the blue curve) had the 590 psi axial compressive stress been applied to it, it does not change the fact that the red curve, which is without the 590 psi axial compressive stress, is the one (the most brittle one) has to be considered for the design acceptance because the same SC wall element in any particular location in the SC wall will be subjected to shear plus axial compression at one time and shear plus no compression at another time and shear plus axial tension at yet another time when the wall swings back and forth, during the entire period of earthquake ground motions. This is why ductility should be **at all locations and under all load conditions** and to guard against **all the unexpected loadings** in addition to the known loading conditions.

Therefore, there is no need to have a prolong discussion on the effect of this 590 psi axial compressive stress on the comparison of the SC and RC elements (or curves) that would be only academic but has no impact on the fact that the SC wall element is" **brittle",** which is not acceptable in concrete structural design in general and should not be accepted for the AP1000 shield building wall in particular.

The DPO panel should and could have resolved or settled this 590 psi axial compressive stress issue immediately after my March 24, 2014 response if it still had doubt with my response.

In order to put an end on the issue of "brittleness" in shear of the SC wall element and the wall, the following excerpts are provided to enhance the understanding of the brittleness of the SC wall element and the wall:

Dr. Vecchio stated in his July 9, 2010 letter (reference 6), "The test beams representing the Shield Building wall detail with [ ] a,c tested in out-of-plane shear (OOPS) at Purdue, exhibited shear-critical **brittle** failures. Westinghouse Electric Corporation (WEC) contends that, even so, the result is acceptable because code-calculated strengths were achieved by the test specimens and that the design shear forces fall well short of these strengths. However, the load contour plots provided by WEC show that, in general, the moment demand-to-capacity ratios are significantly lower than the shear demand-to-capacity values. In other words, for out-of-plane action, shear demand is more critical than

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flexural demand; if the loads were increased in fixed proportion, the wall would sustain a **brittle** shear failure first. This violates the intent of Clause 21.3.4.1.

Dr. Hsu, stated in his Nov. 14, 2010 letter (reference 7) "The tie bars proposed by WEC are size #6 with 17 in spacing in both the directions of height and circumference. However, tests at Purdue University showed that the SC modules failed by out-of-plane shear in a **brittle** manner."

Dr. Varma, who conducted the tests for Westinghouse, stated that he was sure that all the test specimens, [ ] [a,c] (reference 8)

Dr. Vecchio stated in his December 23, 2009 letter (reference 9), "However, finally, there seems to be agreement from WH that **ductility** is an important issue and that the response of the Shield Building should possess sufficient ductility in its response at all locations and under all load conditions." It is important to note that the **ductility** should be **at all locations and under all load conditions.** This statement contradicted the staff's belief and statement "The Code does not specify a ductility level nor does it specify that **ductility** should be in every single structural component of the structure," as mentioned above.

Dr. Vecchio stated in his Juluy 9, 2010 letter (reference 6), "In other words, the design shear force is dictated by the flexural capacity of the member, **ensuring that a ductile flexural failure occur before a brittle shear failure can develop**. In the case of the SB wall, the [ ] [a,c] steel faceplates create a large moment capacity, and thus the shear capacity required to maintain a ductile failure mechanism is high, regardless of the actual out-of-plane shear forces acting."

Dr. Vecchio stated in his July 9, 2010 letter (reference 6), '' Although ACI-349 does not define specific target levels for **ductility**, it is worth noting that it implicitly requires that the failure mechanism be **ductile** regardless of the magnitudes of the actual design loads. Thus, for shear design of flexural members, Clause 21.3.4.1 states that "…the design shear force shall be determined from consideration of the statical forces on the portion of the member between the faces of the joints. It shall be assumed that moments of opposite sign corresponding to the **probable flexural moment strength** act at the joint faces…". In other words, the design shear force is dictated by the flexural capacity of the member, ensuring that a ductile flexural failure occur before a brittle shear failure can develop. In the case of the SB wall, the [ ] [a,c] steel faceplates create a large moment capacity, and thus the shear capacity required to maintain a ductile failure mechanism is high, regardless of the actual out-of-plane shear forces acting."

Dr. Vecchio stated in his July 9, 2010 letter (reference 6), "The test results showed that for the OOPS specimens with span-to-depth (a/d) ratios of 2.5 and 3.5, no significant yielding occurred prior to **brittle** shear failure. Thus, it is not possible to define ductility values based on multiples of the yield deflection. The approach adopted by WEC, defining the yield displacement as the point at which the specimen achieved the theoretical strength ($V_c + V_s$), **is nonsensical and misleading.**

Dr. Hsu stated in his Nov. 14, 2010 letter (reference 7)," More importantly, the requirement of **ductile shear failure** is not intended to guard against the loading assumed in the static pushover analysis. The **ductility** requirement is intended to guard against all the unexpected loadings that are not designed for." The statement clearly indicates that the **shear ductility** is required at all locations to guard against **all the unexpected loadings** in addition to the known loading conditions.

The above excerpts clearly stated that the wall element failed in a brittle manner in shear, and shear ductility in the wall element and wall is required and was agreed upon between the NRC and Westinghouse. This information was either presented or available to the DPO panel. In addition to this information, no one can deny the brittleness of the wall element represented by the red curve, and claim that he has never seen the red curve. Therefore, there is no rational basis that could explain why (1) the DPO panel could form its opinion that the wall element is "ductile", and kept it for more than two years and all the sudden dropped the bombshell revelation on January 22, 2015, and blamed me for not mentioning the flexure test, and (2) the DPO panel would not communicate with me on my responses to its questions related to the comparison of test data between SC and RC elements, and then insisted that its opinion was correct and mine was wrong in the next report while in fact its opinion was wrong and meaningless with respect to the wall design. Unfortunately, these two opinions of the DPO panel had misled Mr. Glenn Tracy into believing that the SC wall was ductile and use it as the bases to reach his decision by stating "Based on my review of information provided by you, the Panel, the NRC staff, and the insights from the meetings described above, I agree with the Panel's conclusion that the SC panels have sufficient strength and ductility."(reference 5)

<u>**Conclusion on the shear ductility evaluation of the AP1000 shield building wall**</u>

There is no doubt that the AP1000 shield building SC wall element and the wall is "brittle" in shear, and that the DPO panel's understanding of the "ductility" or "brittleness" issue is in a state of confusion, and its incorrect opinions has misled Mr. Tracy into believing that the SC wall element and the wall are "ductile"

III. Safety factors for shear in the wall are less than 1.0, which indicates unsafe design

III.1 Function of the shield building wall

The AP1000 shield building consists of a cylindrical wall, supporting a dome on its top, and is anchored down to a concrete basemat. During earthquakes, the cylindrical wall is the only mechanism in the shield building that supports the weight of the dome, and resists the vertical and lateral (horizontal) inertia forces and energy imparted to it by the earthquake ground motions. Therefore, the shield building wall must be strong enough to possess sufficient **strength** to resist theses forces, and **ductile** (tough) enough to swing, or oscillate, back and forth during earthquakes to absorb and dissipate the energy imparts to it.

III.2 Minimum requirements for shield building walls to demonstrate that proper function

Reinforced concrete shield buildings in the existing operating nuclear power plants in the United States were designed to meet American Concrete Institute (ACI) Codes. The older plants used the ACI 318, "Building Code Requirements for Reinforced Concrete", and the newer plants used ACI 349 "Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary." The basic requirements in the two Codes are the same. The ACI 349 Code copied the ACI 318 Code and added loads that ordinary buildings did not design for, such as tornado missiles, to the design requirements. I had personally designed reinforced concrete shield buildings based on ACI Codes prior to my joining to AEC (the predecessor of the NRC). As a result of my interactions with the ACI 349 Code Committee members, and my review of the ACI 349 Code, the Code was endorsed by the industry and the NRC.

III.3 General Requirements in design code, standard, and guide that applicable to shield building wall design

ACI Code is a set of rules specifying desired safety goals, and provides minimum design requirements and upper limits of material strengths for design in an effort to protect public safety, but is not a detailed design procedures nor design aids (Slides 38 and 39 in Reference 1). With respect to the toughness requirement, ACI 318-08 Code commentary states "the design and detailing requirements should be compatible with the level of energy dissipation (or toughness) assumed in the computation of the design earthquake forces." With respect to the strength requirements, ACI Codes specify upper limits of strength (capacity) for each different type of wall failure modes, and these limits were established from test data. These limits imply that a wall reaches one of these limits it may fail or collapse. One of the failure modes in walls is initiated from yielding of steel reinforcing bars in the wall and yielding keeps going until concrete being crushed eventually in the wall, and this type of failure mode is gradual and ductile and can absorb or dissipate a great amount of energy imparts to the wall from earthquake ground motions. Another type of failure mode is the sudden crushing of concrete in the wall while the steel reinforcing bars in the wall have not yielded, or just start yielding, and this type of failure is sudden and brittle and does not absorb or dissipate a great amount of energy. Therefore, walls that are part of seismic resistant systems in a building should be designed to achieve ductile failure modes.

III.4 Specific requirements in design code, standard, and guide that applicable to shield building wall design

III.4.1 ACI Codes' upper limits for shear

ACI 318-1963 Code specified that the shear stress in walls shall be limited to $\mathbf{10\sqrt{f_c'}}$. Section R18.10.4 of the ACI-318-2014 Code states " If the factored shear at a given level in a structure is resisted by several walls or several vertical wall segments of a perforated wall, the average unit shear strength assumed for the total available cross-sectional area is limited to $\mathbf{8\sqrt{f_c'}}$ with the additional requirement that the unit shear strength assigned to any single vertical wall segment does not exceed $\mathbf{10\sqrt{f_c'}}$. The upper limit of strength to be assigned to any one member is imposed to limit the degree of redistribution of shear force."

III.4.2 Purpose of the limits

Section R11.54.3 of the ACI-318-2014 Code states "This limit is imposed to guard against **diagonal compression failure** in shear walls."

III.4.3 Basis for the limits

"Modeling and acceptance criteria for seismic design and analysis of tall buildings" (Reference 10), page 4-8, states "The median ratio of Vtest /Vn was 1.38, with a standard deviation of 0.34, indicating that ACI 318 requirements provided a lower-bound estimate of tested wall shear strength." Vn represents the shear strength calculated by ACI Code's method. I handed the shear test data that I had conducted to Glenn and the DPO panel members during my September 8, 2014 presentation to them. My data of Vtest /Vn fall between 1.0 and 1.33, which match well with those tested by others that had a mean value of 1.38. The data indicated that concrete **diagonal compression failure** had occurred when shear stress went above, **10√f_c'** in individual walls, and therefore the limit of **10√f_c'** represents concrete material crushing (upper limit) which is brittle failure the ACI Code tries to prevent.

III.4.4 Recommended shear strength limits for design of structural (shear) wall over the years

ACI 318-11 Building Code states: "As the name implies, "Building Code Requirements for Structural Concrete" is meant to be used as part of a legally adopted building code and as such must differ in form and substance from documents that provide detailed specifications, recommended practice, complete design procedures, or design aids." Therefore, documents that provide detailed specifications, recommended practice, complete design procedures, or design aids for structural walls that resist shear become the responsibility of the designer and reviewer for the walls.

After the 1985 Chile earthquake that caused wall damages and collapses, the United Stated National Science Foundation sponsored a US-Chile research program. The result of that program limits the maximum shear stress to **6√f'c'** for wall design (slide 41 in reference 1).

After the 1994 Northridge earthquake, which caused 57 death, 8,700 injured, and $20 billion property damage, the Uniform Building Code that governed the Western States of USA, including California, revised its Code in 1997 to limit the wall axial compression force, P_u, to be equal to, or less than, $0.35P_o$, where P_o is the axial strength at zero eccentricity, and FEMA (Federal Emergency Management Agency) – provided financial support to NEHRP (National Earthquake Hazard Reduction Program), and ASCE (American Society of Civil Engineers) to develop standards for evaluating safety of existing buildings and design guides for new buildings. In 2000 FEMA and ASCE published FEMA 356, "Seismic Rehabilitation Prestandard" (slide 47 in reference 1), which states:

a) "In general, higher axial load stresses and higher shear stresses will reduce the flexural **ductility** and **energy absorbing capability** of the shear wall", and thus

b) "Shear walls or wall segments with axial loads greater than 0.35 Po shall **not be considered effective** in resisting seismic forces," and

c) "For shear walls and wall segments where inelastic behavior is governed by shear, the axial load on the member **must be ≤ 0.15 *Ag*** f_c', the longitudinal reinforcement must be symmetrical, and the maximum shear demand **must be ≤ 6√f_c';** otherwise, the shear shall be considered to be a force-controlled action."

The wall is considered to be a ductile wall only if the conditions of axial vertical compressive stress is **less than or equal to 0.15 f_c'** and shear stress demand is **less than or equal to 6√f_c'** are satisfied. If not, the wall is considered to be non-ductile.

The 2010 Chile earthquake, and the 2011 Christchurch, new Zealand earthquake had caused wall damage, collapse, and buckling. As a result, the "Seismic Design of Cast-in-Place Concrete Special Structural Walls - A guide for Practicing Engineers" by NEHRP (National Earthquake Hazards Reduction Program), ATC (Applied Technology Council), CUREE (Consortium of Universities for Research in Earthquake Engineering), and was published by NIST (National Institute of Standard and Technology), March 2012. The Design Guide States:

> "Although ACI 318 permits factored shear on individual wall segments as high as Vu = 10φ√f'c Acv, the flexural ductility capacity for such walls is reduced compared with identical walls having lower shear. This Guide recommends factored shear, calculated considering flexural **overstrength** (see Section 3.1.3), not exceed approximately **4φ√f'c Acv to 6φ√f'c Acv** so that flexural **ductility capacity** is not overly compromised." (see page 7 in reference x)

The recommended shear strength for wall design has been decreased from **6√f_c'** in 1965 to **4√f'c to 6√f'c** in 2012. This reduction was not only due to the observation of wall failures and collapses during earthquakes, but also due to the scientific research conducted by Dr. Vecchio at the University of Toronto, Canada, and Dr. Hsu at the University of Houston, Texas, by using the only two most sophistic testing equipment in the world for wall panels (slides 20 through 35 in reference 1). The testing and research results indicated that shear alone can cause wall concrete material into compression failure, and the maximum compressive strength in walls could be as low as 20% of the concrete compressive strength, fc' (slides 21 and 22 in reference 1). This scientific research results explain the three recommendations (a), (b), and (c) for wall design in FEMA 356 Prestandard, and the shear strength of **4√f'c to 6√f'c,** depending on the magnitude of compressive stress, in the 2012 NEHRP wall design guide.

As a result of wall buckling and compression failure in the 2010 Chile earthquake, and the 2011 Christchurch, new Zealand earthquake and additional test results in the laboratory, the ACI 318-2014 version Code added a new requirement for the thickness of walls in the boundary element region to be not less than h_u /16, where h_u is the laterally unsupported heights at extreme compression fiber of wall. This provision is to provide sufficient wall thickness to lower the compressive stress in the wall to avoid wall buckling due to slenderness problem.

III.4.5 Shear strength values for SSE and RLE that should be used for AP1000 shield building wall design or evaluation

The AP1000 shield building wall is subjected to out-of-plane shear and in-plane shear simultaneously during earthquake, and recent research has established that the interaction will reduce the strength for both (slide 57 in reference 1). The AP1000 shield building wall is also subjected to compressive stress due to the heavy PCS tank that no other shield building is. As a result of these two factors, the proper shear stress level for the design basis earthquake, SSE, should not be higher than **4√f'c,** and the shear strength for the RLE earthquake should not be higher than the ACI Code' limit, **8√f'c.**

Appeal to EDO
John Ma
March 31, 2015

III.4.6 Safety factors in shear for the AP1000 shield build

As shown in the diagram below, the shear stress for the loading conditions including SSE has exceed [ ] [a,c] and the shear stress for the loading conditions including RLE has exceed [ ] [a,c] The exceedance of shear stress for the loading conditions for both the SSE and RLE cases is by a factor of about two (2). Therefore, the safety factors for [ ] [a,c] which is less than 1.0. As stated in step 8 of the design process on page 2 of this report, the thickness of the wall should be increased and then repeat the design process until the safety factors reach 1.0 or greater. The check of safety factor for shear is a necessary step in the design process for building designer and the review process for the NRC reviewers.

a,c

In fact that Westinghouse had under predicted shear stress in the wall. as represented by the green curve, because (1) the irregular zigzag shape connections between the SC wall and the RC wall, as shown on page 12 of my DPO, was not properly represented in the mathematical model, (2) code required accidental torsion was not performed (see page 13 of my DPO), and (3) code required P-delta analysis was not performed (more detail discussion later). Had those three items been properly included in the Westinghouse' mathematical model, the shear stresses in the wall would definitely be greater than those values represented by the green curve (for SSE) and red curve (for RLE), and thus further reduces the safety factor in shear [ ] [a,c]

III.4.7 The DPO panel's evaluation on shear strength of the AP1000 shield building is incorrect

Mr. Glenn Tracy states in his letter to me (reference 5), "Section 4.1.2.4 of the Panel's report provides a discussion of the strength of the SC panel, quoting the Panel's consultant, Mr. Loring Wyllie, as follows:

> "…the in-plane shear capacity of the SC wall should have been calculated considering the [ ] [a,c] For in-plane shear, the ACI 349 equations are probably not directly applicable as the [ ] [a,c]

> [ ] a,c provide for more shear capacity than a normal reinforced concrete wall. Using ACI 349, shear capacity would be limited to 8√fc'hd by Chapter 21. WEC used a [ ] a,c The true in-plane shear strength is much greater and dependent on the [ ] a,c provide bond of the steel plates to the concrete core. "

The DPO panel's evaluation on the shear strength of the AP1000 shield building relied on Mr. Loring Wyllie's opinion and judgment. In the above statement, Mr. Wyllie stated (1) he believed that the Westinghouse' use of [ ] a,c was conservative due to the use of [ ] a,c while recognizing ACI Code's limit is 8√fc', and (2) the true in-plane shear strength is much greater and dependent on the [ ] a,c provide bond of the steel plates to the concrete core.

Mr. Wyllie is right on the influence of tie-rods on the shear strength, as was demonstrated in the tests that the use of [ ] a,c (the red curve exhibited much lower shear strength, as well as ductility, compared to that of the [ ] a,c which is basic knowledge in concrete shear. That was the reason that Dr. Vecchio, Dr. Hsu, and I requested Westinghouse to test the elements that it chose for use in the AP1000 shield building wall and obtain their shear strength as well as ductility to compare with that of RC elements and then determine whether the use of ACI Code was appropriate and safe for the design of the AP1000 shield building.

Should the NRC rely on Mr. Wyllie's belief that the Westinghouse' use of [ ] a,c was conservative, and accept that value instead of the ACI Code's limit of 8√fc', which is the design basis of the AP1000 shield building? My answer is "No", because the Code' limit of 8√fc' is not only a legal limit, but has technical reasons of considering "stability" of the wall and its severe consequence on the collapse of the building if the wall fails. **As stated in Section 3.4 of this report, the diagonal compression failure could occur after the shear stress reaches [ ] a,c and this is a concrete material failure which is brittle, regardless how much steel reinforcement is in the SC or RC elements.**

The discussion of the previous paragraph is on the technical and legal limit of the [ ] a,c What was Mr. Wyllie's belief that the recommend shear strength for the design of structural (shear) walls should be? As stated in Section 3.4.4, the most recent recommended shear strength for the design of walls is in the "Seismic Design of Cast-in-Place Concrete Special Structural Walls - A guide for Practicing Engineers" by NEHRP (National Earthquake Hazards Reduction Program), ATC (Applied Technology Council), CUREE (Consortium of Universities for Research in Earthquake Engineering), and was published by NIST (National Institute of Standard and Technology), March 2012. The recommended shear strength values are from 4√fc' to 6√fc', and Mr. Wyllie was one of the three people in the review panel of this design guide.

In slides 75 through 81 of reference 1, I presented how NRC/NRR had averted a major mistake created by SQUG, SSRAP, URS, and the NRC/RES. During my oral presentation to Mr. Tracy and the DPO panel, I mentioned that Mr. Wyllie was one of the three SSRAP members, who were hired by SQUG to direct, review, and approve anchorage criteria for resolving the USI A-46 issue, and I found that the original criteria was deficient by a factor of 2. I used that information as an example to show that anyone can make mistakes, and urged the DPO panel to carefully examine whether Mr. Wyllie's opinion was correct or not, and should not blindly trust in what he had believed.

The questions in front the DPO panel and the staff are (1) if the panel believes Mr. Wyllie’s belief that [ ][a,c] is a conservative value for the shear strength of the AP1000 shield building, should the panel also believe that Mr. Wyllie’s belief that the shear strength values of 4√fc’ to 6√fc’, as stated in the design guide, which was reviewed by him, should also be used for the design of AP1000 shield building wall for the loading condition including SSE, (2) does the panel and the staff still believe that Mr. Wyllie’s belief of [ ][a,c] should be used for the evaluation of the AP1000 shield building wall or that the ACI Code’ value of 8√fc’, as I have argued from legal and technical points of view, (3) how can the DPO panel justify its statement that the AP1000 shield building wall has sufficient shear strength to resist RLE and meet the NRC seismic margin requirement while the shear stress has significantly exceeded the [ ][a,c] as presented in the diagram of my DPO and also in the diagram above? and (4) both the NRC staff and the DPO panel did not provide their design or evaluation process and acceptance criteria for the AP1000 shield building wall that had been, or should have been, used as the basis for their evaluation and conclusion that the AP1000 shield building is safe, and therefore their statement and conclusion have no technical basis, only opinions similar to the bridge designer had in the ancient Rome time.

III.4.8 Modeling errors of the AP1000 shield building that resulted in less shear stress

As stated in my DPO and in Section 3.4.6 of this report, there are three modeling errors: (1) the irregular zigzag shape connections between the SC wall and the RC wall, as shown on page 12 of my DPO, was not properly represented in the mathematical model, (2) code required accidental torsion was not performed (see page 13 of my DPO), and (3) code required P-delta analysis was not performed.

If anyone submits the AP1000 shield building as an industrial building to be built in Los Angeles, California, because of its height, a three-dimensional mathematical model must be constructed to include (1) its irregular zigzag shape and stiffness of connection to the RC wall below, (2) the auxiliary building roof to be supported by the RC wall, and (3) P-delta effects (reference 11). This mathematical model will have to be subjected to two earthquake ground motion intensities: Design Basis Earthquake ground motions and Risk Targeted Maximum Considered Earthquakes ground motions, similar to the NRC’s SSE and RLE.

Since the SC portion of the AP1000 shield building is actually supported by, and anchored to, the RC wall below, the stiffness of the connection may have significant effects on the dynamic analysis results with respect to frequencies and shear stress of the SC wall, especially when the shield building has significant irregularity, and this stiffness effect was not captured by Westinghouse model.

Both the staff and the DPO panel have dismissed the P-delta effects by offering their calculated deformation values as insignificant due to the P-delta effects. I believe that the P-delta effects in such an irregular building are so complicated, and the only way to obtain the true effect is to include them in the three-dimensional mathematical model, as has been practiced by structural engineers and required by the Los Angeles’ building department, as well as building department in other cities.

With respect to the accidental torsion, unless an analysis is performed no one can be certain what the significant irregularity of the AP1000 shield building could affect the magnitude and distribution of shear stresses in the wall.

Without the above analyses, one can only conclude that the magnitude of shear stresses in the wall, as represented by the green curve in the above diagram, is less than the actual shear stress values in the wall.

III.4.9 Definition of seismic margin and the proper way to attain it

Assuming the AP1000 shield building is to be built in Los Angeles, California, as stated in the previous section, the mathematical model of the building will have to be subjected to two earthquake ground motion intensities: Design Basis Earthquake ground motions and Risk Targeted Maximum Considered Earthquakes ground motions, with the intensity of the latter earthquake ground motion being 1.5 times that of the design basis earthquake ground motions, similar to the intensity of the RLE being 1.667 times the SSE.

The actual analysis process can be seen in slides 29 thru 31 in reference 1. Slide 29 indicated that the intensity of the earthquake ground motion at PGA=0.8g produced initial reinforcement yielding (initial steel yielding is the upper design limit for SSE that ACI code requires.) Slide 30 indicated that at PGA=1.2g, the wall concrete was cracked and steel keeps yielding. Slide 31 indicated that at the intensity of the ground motion at PGA=1.6 the wall concrete was crushed and failed. Therefore, the definition of the seismic margin is the ratio of the intensity of the ground motions when the wall failed to the intensity of the ground motions of the design earthquake basis. For this test wall, the seismic margin is 1.6g/0.8g = 2.0. The reason that this test wall can have a seismic margin of 2.0 is because it was designed to behave as a ductile wall to reach a seismic margin of 2.0. Such an analysis can be performed with confidence because decades of research, including testing, on RC wall elements resulted in methods for estimating shear stiffness and ductility values for RC elements that are necessary to be used for predicting the entire building behaviors during earthquakes (slide 35 in reference 1.)

However, such an analysis was not done for the AP1000 shield building and cannot be done because the shear stiffness and ductility values for SC elements are unknown (slide 36 in reference 1), and therefore no computer code can be developed for use to predict the behavior and failure of the SC wall elements and the AP1000 shield building wall. Consequently, there is no way of knowing the true value of the seismic margin for the AP1000 shield building wall. The safety factor approach in the ACI Code, as described above, can be used to evaluate the seismic margin for the AP1000 shield building wall. Since the design of the AP1000 shield building is strong in bending and very brittle and weak in shear, therefore the building will fail in shear long before it reaches bending failure. This is the result of the Westinghouse design. When the shear stress at SSE has already exceeded the shear strength of the ACI Code's limit of 8√fc', [ ] a,c , the concrete material crushing strain, there is no margin left that the building can sustain a much higher shear stress produced by the RLE, which is 1.667 times the SSE - a common sense approach. Had the Westinghouse used the recommend design method stated in the "Seismic Design of Cast-in-Place Concrete Special Structural Walls - A guide for Practicing Engineers", which is **4√f'c to 6√f'c** for shear strength, then there would have been margins of safety factor in shear of 8/4=2.0 for (**4√f'c)** , and 8/6=1.33 for (**6√f'c)** available for the increased shear stress generated by the RLE. The use of a design shear strength of **4√f'c** for the SSE would give a seismic margin of 1.667 that is required by the NRC seismic margin analysis. Since Westinghouse chose a design shear strength of

[ ] [a,c] for the SSE condition, it is obvious that the design would never be able to satisfy the RLE condition and the NRC seismic margin requirement. By understanding the definition of seismic margin and the proper way of attaining it, it can be understood that the method used in the DPO Panel report for calculating the seismic margin for the AP1000 shield building wall was incorrect because those numbers can be and were manipulated to justify the wall meeting the NRC seismic margin requirement.

<u>Conclusion on the shear strength evaluation of the AP1000 shield building wall</u>

While the ACI Code provided the limit of average shear strength, 8√fc' , for concrete walls in a building with the requirement that each individual wall shall not exceed, 10√fc', as the legal limits, and that the safety factor, the ratio of strength to stress, shall be greater than 1.0 as the basis of safety acceptance, the staff and the DPO panel provided no shear strength values and safety factors for the evaluation on the safety of the AP1000 shield building wall, except stating that the wall is safe which has no technical basis.

VI. While I believe that the staff's and the DPO panel's conclusion that the aircraft missile cannot penetrate through the SC wall is illogical, the DPO panel believes its conclusion logical

VI.1 Brittle wall lacks energy absorption capability for missiles

Since the AP1000 wall element failed in a brittle manner (the red curve) during the shear test (a force perpendicular to the surface of the element similar to the aircraft missile to the wall surface), I realized immediately that the wall would have problem in its energy absorption capability to resist the energy imparts to it from the aircraft missiles and in punching shear strength to resist missiles. Slide 69 in reference 1 presents a table that listed the required concrete wall thickness for not being punched through by aircraft in nine analyses. Except the AP1000 shield building wall thickness of three feet, all other walls thicknesses are about six feet in that list, as an indication that the three feet AP1000 shield wall is insufficient to resist the aircraft missiles. I used the fact that the punching shear strength of a concrete slab, footing, and wall is related to the square power of their thickness, and stated that the three feet thick of the AP1000 shield building wall has about only ¼ of the punching shear strength that the six feet thick shield building wall has, as another reason to doubt the adequate punching shear strength of the AP1000 SC wall. However, the DPO panel stated that it was the curvature in the AP1000 shield building that is responsible for the great aircraft missile impact resistant comparing with the flat walls or slabs. I then submitted a paper to Glenn and the DPO panel from the ACI journal which concluded that the curvature in the curved beam would reduce the shear strength compared with a flat beam. The DPO panel explained that the tests done for the curved beams were simply supported while Dr. Rashid's mathematical model was an arch with fixed end supports, and therefore the results are opposite due to the support conditions. The fact is that the shield building wall in the circumferential direction is neither simply supported nor fixed end supported, and its fixity is some value in between, and therefore the DPO panel's explanation is not convincing.

Since the DPO panel insisted on the curvature issue as its main reason to justify the adequacy of the wall thickness for AP1000 shield building, and even overrides the reduction of punching shear strength by a factor of four due to its one-half of the thickness of the EPR wall thickness, I countered with the reason that the EPR shield building wall is not flat, and has curvature. The DPO panel stated on page 2-9 of its January 22, 2015 meeting report, "Meeting summary section 3.5.2.3 addresses the Submitter's comparison of the EPR shield building to the AP1000, noting that the AP1000 building has a smaller diameter which increases the beneficial effects of arching action." Can the small amount of curvature increase from the EPR to AP1000 shield building really increase the so called "arching action" so much to push the punching shear strength up to a factor of more than four to overcome the thickness deficiency? Reference 12 conducted an aircraft impact analysis on an innovative NPP reactor (IRIS) containment building, which has a diameter of about 88 feet, less than the 145 feet diameter of the AP1000 shield building wall, and has much more curvature than that of the AP1000 shield building, and concluded that the missile penetration depth is about 1 meter, which is more than the AP1000 shield building wall thickness. Therefore, the DPO panel's reason that the curvature effect of the AP1000 was responsible for its not being penetrated by aircraft missiles is not convincing.

On the missile penetration issue, we know that the thickness of the wall is the most important variable because the punching shear strength is proportional to the square power of the wall thickness, and the next important variable is the wall's energy absorption capability, which is measured by the shear ductility characteristic of the wall element and the wall because it needs to balance the energy imparts to the wall by missiles. The wall could fail in a flexure (bending) mode, a punching shear mode, or a combined flexure and shear mode depending on its design. Since we already know that the SC wall is brittle in shear, and strong in bending, the SC wall is going to fail in shear prior to its failure in bending if impacted by missiles. This is why Westinghouse only examined the punching shear strength for the wall when it conducted the automobile missile event, and the NRC reviewer only reviewed the punching shear issue, as stated in the NRC SER Section 3.5.1.4.2 below:

> "In the event of an automobile missile strike on the nuclear island structures 9.14 m (30 ft) above grade, there would be two safety concerns for the seismic Category I structures: (1) local damage; and (2) global damage. The staff reviewed the analysis of local damage in APP-1000-CCC-015, Revision 0 entitled: "Nuclear Island-Tornado Missile Automobile Impact 30' Above Grade." In the report, the applicant considered an impact area 2.01 m by 1.31 m (6.6 ft by 4.3 ft) by the automobile missile with a shear area 0.39 m x 0.60 m (1.29 ft x 1.98 ft) at the weakest location. The shear resistance of the RC wall was assessed at 112.99 pounds per square inch (psi), and the maximum shear stress induced by the impact was calculated to be 89.15 psi. Since the applied shear stress is less than the concrete wall shear resistance, the applicant concluded that the wall is able to resist the impact from being punched through. On this basis, the staff considers that the local damage concern at the impact spot is resolved. Another local damage concern is the crack initiation at the siding missile strike site. If the site is located at a critical section, the crack may grow unstably under the maximum stress induced by the automobile missile impact force as well as the strong tornado wind load. This safety concern was addressed in Section 3.3.4."

The DPO panel stated that both Westinghouse and the staff had performed aircraft impact analyses on the SC wall of the AP1000 shield building and used the same computer code. In addition to these two analyses, Dr. Rashid had model a portion of the SC wall as an arch with unyielding supports. The analyses results indicated the failure mode was bending instead of (punching) shear. The results suggest that the (punching) shear failure mode had not been built into the computer or incorrectly built into the computer. By looking at the design of the AP1000 shield building, the shear strength and shear energy absorption capability were much weaker or less than that of bending (flexure) of the SC wall, and therefore the wall should have been failed in shear mode instead of bending mode. The shear test with a force acting perpendicular to the surface of the SC wall element resulted in brittle failure (the red curve), and this curve should have been built into the mathematical model in the computer, and then the analysis result would be a (punching) shear failure mode.

Conclusion on the aircraft impact evaluation of the AP1000 shield building wall

Based on the design of the AP1000 shield building wall, the wall is very strong, with great amount of energy absorption capability, in bending (flexure) but very weak, with small amount of energy absorption capability, in shear. The actual local failure mode of the wall, as a result of aircraft missile strike, should be (punching) shear instead of bending. However, the analyses results from the computer codes indicated a bending failure mode, suggesting that the shear failure mode (the red curve) had not been, or had been incorrectly, coded into the computers.

References

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2. “Summary of January 22, 2015, Meeting Regarding Differing Professional Opinion (DPO) 2012-002” from Joseph F. Williams, senior Project Manager, to DPO Case File, Differing Professional Opinion 2012-002, February 23, 2015
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4. Review of the differing Professional Opinion (DPO) Panel Report Regarding Structural Integrity Concerns with the AP1000 shield Building (DPO-2012-002) by John S. Ma, Ph.D., P.E., March 27, 2014
5. “Decision Regarding Differing Professional Opinion Involving Safety of The AP1000 Shield Building (DPO-2012-002)” from Glenn M. Tracy, Director, Office of New Reactors to John S. Ma, Senior Structural Engineer, Division of Engineering, Office of New Reactors, February 27, 2015
6. “DUCTILITY IN SHEAR RESPONSE OF SHIELD BUILDING,” FRANK VECCHIO, JULY 9, 2010
7. “Comments on the safety of AP1000 Shield Building,” by Thomas T.C. Hsu and Y.L. Mo, Nov. 14, 2010
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9. “Re: NRC-WH Technical Clarification Meeting of December 21-22. 2009,” by Frank J. Vecchio, Ph.D., P. Eng., December 23, 2009
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12. "Preliminary Evaluation of Aircraft Impact on a Near Term Nuclear Power Plant," by R. Lo Frano and G. Forasassi, International Conference Nuclear Energy for New Europe, 2009

Document 8: Office Manager’s Statement of Views

July 31, 2015

MEMORANDUM TO: Mark A. Satorius
Executive Director for Operations

FROM: Glenn M. Tracy, Director ***/RA/***
Office of New Reactors

SUBJECT: STATEMENT OF VIEWS REGARDING APPEAL OF DIRECTOR'S DECISION FOR DIFFERING PROFESSIONAL OPINION 2012-002

This memorandum provides my statement of views regarding the appeal of Differing Professional Opinion (DPO) 2012-002,[1] per the requirements of Management Directive 10.159.[2] After review of the appeal, and consultation with the ad hoc DPO Panel and NRC staff experts, I have concluded that the appeal does not provide any new information or new interpretation of existing information that changes the positions described in my Director's Decision.[3] The basis for this conclusion is documented in a summary of a meeting held on June 17, 2015,[4] where the issues described in the appeal were thoroughly discussed.

cc: G. Holahan, NRO
E. Hackett, ACRS
[redacted] NMSS
[redacted], NRO
J. Tappert, NRO
R. Caldwell, NRO
J. Steckel, NRO
J. Williams, NRO
J. Ma, NRO

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1 "Appeal to EDO Regarding Director's Decision on DPO 2012-002," March 31, 2015.

2 MD 10.159, "The NRC Differing Professional Opinions Program," May 16, 2004, ADAMS Accession No. ML041770431.

3 "Decision Regarding Differing Professional Opinion Involving Safety of the Ap1000 Shield Building (DPO-2012-002)", February 27, 2015, ADAMS Accession No. ML15062A345.

4 "Summary of June 17, 2015, Meeting Regarding Appeal of the Director's Decision for Differing Professional Opinion 2012-002," July 30, 2015, ADAMS Accession No. ML15195A410.

Document 9: DPO Submitter's Briefing to EDO

U.S.NRC
United States Nuclear Regulatory Commission
Protecting People and the Environment

AP1000 Shield Building is Unsafe

Does not meet New Rules (Seismic Margin Analysis and Aircraft Impact Analysis) Requirements

An APPEAL to EDO

by
John S. Ma, Ph.D., P.E., Charter Member of NRC
Registered Professional Engineer in Maryland, Virginia, and California
Member of ACI and ASCE
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NRO/DE/SEB
June 21, 2016

Contains Proprietary Information

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Information Referred to NRO or DPO Panel is More than One Year Old

Information referred to the NRO or the DPO Panel in this presentation is prior to my appeal letter to the EDO, dated March 31, 2015, because I have neither been contacted by, nor received any questions from, them or the EDO since then. Therefore, I would be happy to address any questions the EDO may have that were arisen from the discussion between the EDO and the NRO or the DPO panel in the past year

Falsehood Fooled NRO Truth Frees EDO

1. Concrete buildings are designed or evaluated in accordance with American Concrete Institute (ACI) Code's requirements that reflect **<u>true</u>** structural engineering principles or laws of physics
2. However, the DPO panel totally disregarded ACI Code's requirements that are practiced by structural engineers by cherry picking information and inventing its own theories and solely relied on its own opinions and logics that are **<u>false</u>**
3. Since ACI Code is the design/licensing basis for the AP1000 shield building, the EDO is urged to take a hard look at the validity and legality of opinions from the DPO panel and see the distinction between **<u>falsehood</u>** and **<u>truth</u>** in structural engineering principles
4. Differentiating the **<u>falsehood</u>** from the **<u>truth</u>** will set the EDO free to lead the unsafe AP1000 shield building design to safety

Goal for New Reactor's Structural Design

1. The worst accident for nuclear power plants is the **early** and **large** release of radioactive materials into the atmospheres
2. Therefore, the goal for the design of structures is to prevent the **breach of containment** and **core damage**
3. Due to the discovery of increasing seismic activities and intensity worldwide and the September 11, 2001 event of a large aircraft impacting the World Trade Center caused its collapse, the new design for the shield and containment buildings have been upgraded worldwide
4. New rules were introduced to achieve this goal

New Rules to Achieve this Goal

- NRC's New Rules for New Reactor Structural Design that were not applied to the design of existing nuclear plants

1) Existing shield building structural design only considered Safe Shutdown Earthquake (SSE)
2) New shield building structural design shall consider Review Level Earthquake (RLE) = **1.67** x SSE, as a result of the NRC Seismic Margin Analysis (SMA) rule
3) Aircraft impact Analysis (AIA) rule

   A large commercial airplane impacts on a new shield building that was not designed for existing shield buildings in operating plants

- Other Countries have similar new rules

Result: Higher seismic and aircraft impact loadings should result in stronger (thicker) and more ductile (energy dissipation/absorption capability) shield building walls than that in operating plants

Wall Thickness Comparison between AP1000 Shield Building and other New Shield Buildings

Excerpt from the “Workshop of the INPRO Dialogue Forum on Nuclear Energy Innovations,” 1-4 February 2010, IAEA, Vienna, Austria

Selected features of current PWR designs

	AES-2006	APWR 1700	AP1000	EPR
Thermal/ electric power	3200/1150 MW	4451/1700 MW	3415/1200/1117 MW	4250/1655 MW
Design characte-ristics	Evolutionary design, combination of active and passive features, 4 loops, horizontal SG	Evolutionary design, limited passive features, vertical SG	Extensive passive safety features, simplified construction and operation, 2 steam generators, 4 cold legs, vertical SG	Evolutionary design, limited passive features, 4 loops, vertical SG
Reactor coolant system	RCS pressure 16,2 MPa, reactor outlet 328,9°C, steam pressure 7 MPa	RCS pressure 15.5 MPa, reactor outlet 325°C	RCS pressure 15.5 MPa, reactor outlet 321°C. steam pressure 5.76 MPa	RCS pressure 15.5 MPa, reactor outlet 330°C, steam pressure 7.63 MPa
Primary containment	Prestressed concrete with metallic liner, 1100 - 1200 mm, volume 74200 m3, design pressure 0.5 MPa, leak rate 0.2 vol.%/ den	Single prestressed concrete with metallic liner, 1120-1320 mm , design pressure 0.57 MPa	Freestanding, cylindrical steel vessel 44.4 mm thick, volume 56600 m3, design pressure 0.5 MPa, leak rate 0.10 vol.% /day	Prestressed concrete with metallic liner, 1300 mm wall, volume 80 000 m3, design pressure 0.55 MPa, leak rate 0.3 vol. %/day
Secondary containment (Shield Build)	Reinforced concrete, 1,8 – 2,2 m, active annulus venting	Vented surrounding structures in the area of penetrations	Reinforced concrete building with conical shell, open to the atmosphere at the top, 0.91 m thick wall	Reinforced concrete, wall thickness 1.8 m, active venting of the annulus
Plant lifetime	60 years	60 years	60 years	60 years

Note: AP1000 shield building wall thickness (0.91 m ≅ 3 feet) is about ½ of that of AES-2006 (1.8 – 2.2 m) and EPR (1.8 m ≅ 6 feet)

Wall Thickness Comparison between AP1000 Shield Building and other New Shield Buildings (continued)

Source: Information provided to the NRC from the Canadian Nuclear Regulators

<u>New shield building wall thickness</u>

German Convoi NPPs, EPR, TMEA1, and EC6 ≅ 6 feet

AP1000 ≅ 3 feet

Design/Licensing Basis of AP1000 Shield Building

American Concrete Institute (ACI)

Building Codes

Comparison of Design Results between AP1000 and EPR

(Both are designed to same NRC rules, same seismic intensity, and same ACI Code)

1. AP1000 and EPR were submitted to, and reviewed by, the NRC
2. Both shield buildings were subjected to the same rules (SMA and AIA), the same seismic intensity of 0.3g, and the same ACI Code as design/licensing basis
3. In addition to item 2 above, the PCS tank on top of the AP1000 shield building generates additional shear stress during earthquakes, due to its mass, and compression stress, due to its weight, in the wall compared to that of EPR because it does not have a PCS tank
4. In addition to items 2 and 3 above, the AP1000 shield building wall has an irregular zig-zag shape of anchorage (see next slide) that generates additional shear stress during earthquakes in the wall compared to that of EPR because it does not have an irregular shape of anchorage
5. The higher shear stress in AP1000, should have resulted in a thicker wall, because shear strength is directly proportional to its wall thickness. However, the three (3) feet thick AP1000 shield building wall is one-half (½) of the six (6) feet thick EPR shield building wall. This is abnormal in structural engineering design.
6. For identical design rules, criteria, and code, the symptoms (wall thickness) listed above indicate structural design mistake of AP1000 shield building wall

Cause for Additional Shear Stresses

(The Irregular Shape of Anchorage)

Existing or new shield building walls, such as EPR, have a perfect cylindrical shape from top to bottom, but the AP1000 shield building wall has an irregular zig-zag shape in both vertical and horizontal directions as shown below

[ ]a,c

Result: 1. This highly irregular shape of connection or anchorage both in the horizontal and vertical directions generates additional shear stress in the wall during earthquakes due to torsion that existing or other new shield building walls are not subjected to

2. The analysis and design of this irregular shape of anchorage are difficult and were not adequately performed

Four Collapsed Stories and Significant Wall Damaged due to Torsional **SHEAR** Caused By Building **Irregularities**

(Note: AP1000 Shield Building is a highly Irregular structure)

Torre Higgins Building: The 20-story office building in Concepcion's city center had significant damage, including four stories that collapsed starting on the 12th floor. Significant damage was also observed in the walls and concrete piers in the stories below the collapsed floors. It appeared that the building experienced unique dynamic effects resulting from the building's irregularities and torsional movement

The Higher the Shear Stress, the Thinner the Wall?

1. In Concrete structural design, the higher the shear stress, a thicker shear wall is required
2. However, the AP1000 shield building wall should have higher shear stress than that of EPR wall as presented above, but AP1000 shield building wall is thinner by a factor of about two (2) to that of EPR
3. This is an indication of design mistakes

For The Higher Risk, a Thinner and Weaker Wall is Provided?

1. The possible failure of walls or the PCS tank anchorage resulting from earthquakes or aircraft missile impact may cause the heavy PCS tank tumbling down and crushing the steel containment and causing **containment breach** and **core damage** while EPR does not have this possibility or risk because it does not have a PCS tank
2. Therefore, due to the risk of "**containment breach**" and "**core damage**", the design result of AP1000 shield building wall should have a stronger (thicker) and/or more ductile (energy dissipation/absorption capability) than that of the EPR. However, the structural design result shows the opposite by a factor of about two (2) in wall thickness
3. Such a large discrepancy in the wrong direction is a clear sign of design mistakes

Major Design Mistakes of AP1000 Shield Building Wall

1. For earthquake design, it violates ACI Code's Requirements

   A. No brittle wall element with insufficient energy dissipation capability

   (the majority of wall elements are brittle with little energy dissipation capability) - **<u>unsafe</u>**

   B. Wall element should possesses sufficient shear strength

   (the wall element's shear strength is deficient by a factor of two) - **<u>unsafe</u>**

2. For aircraft impact analysis, the brittle wall element shear force – deformation relationship property was not included in the analysis, and thus the weak punching shear failure mode was eliminated from the analysis, and only the ductile flexural mode was calculated and that value is **<u>false and unsafe</u>**

ACI Code's Requirements

1. No brittle structure or its component is allowed, and ductility is required commensurate with earthquake force or missile impact force and the importance of buildings
2. New types of construction requires testing to establish its Code's applicability (AP1000 shield building wall is a new type of construction)
3. A structure should be designed to behave and eventually fail in a ductile manner, such as flexure (bending), to provide sufficient energy dissipation and absorption capability and avoid all brittle failure mechanisms, such as shear or compression failure modes

ACI Code's Ductility Requirement for Shell design

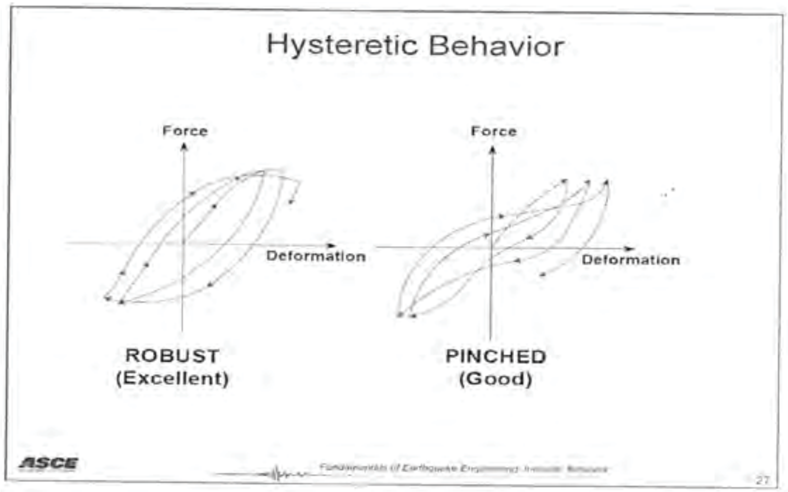
1. AP1000 shield building wall is a cylindrical concrete shell structure
2. ACI 318-08 Code, Chapter 19-Shell and Folded Plate Members, Section 19.4.5, states

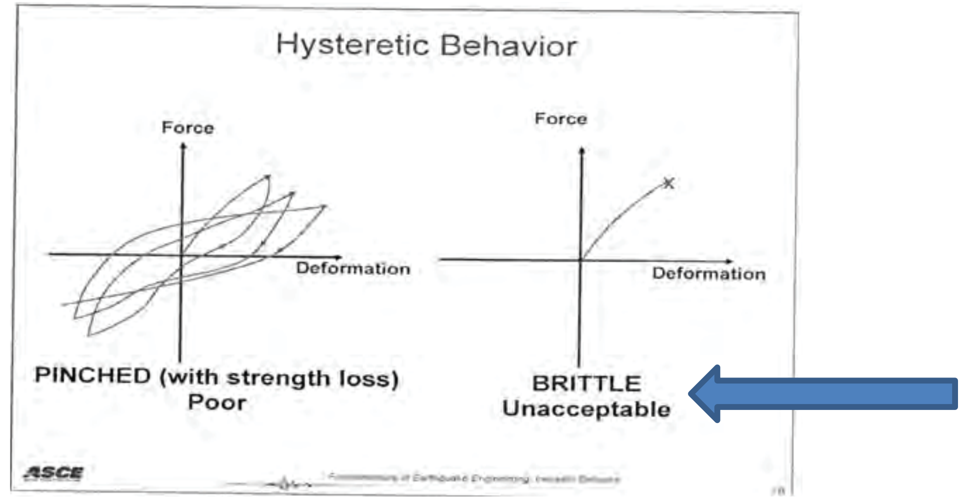
   "The area of shell tension reinforcement shall be limited so that **the reinforcement will yield before either crushing of concrete in compression or shell buckling can take place**."

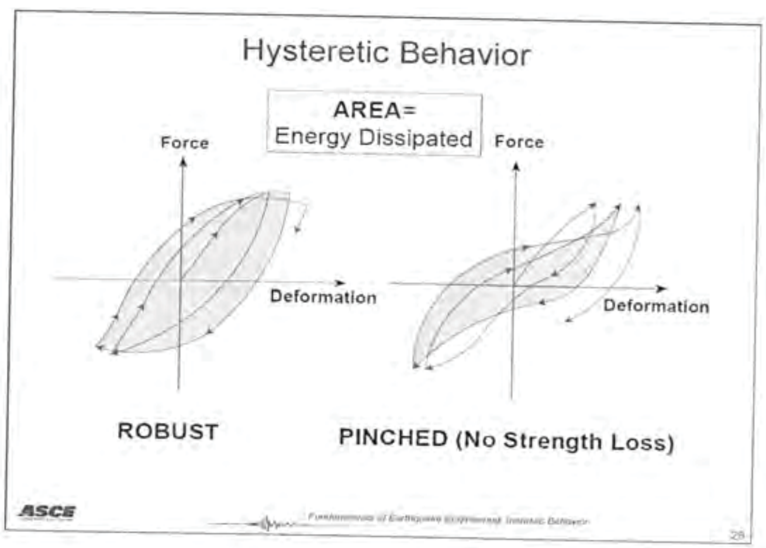
Note: The steel plates in the AP1000 shield building wall is the tension reinforcement as referred to here in the ACI Code, and the steel area is more than that of ordinary reinforced concrete shell. Therefore, design to avoid concrete crushing due to shear (generated by earthquakes) plus compression (due to the weight of PCS tank and water) become important

Brittle Structural Element is unacceptable and Ductility is Required

(Excerpts from 2016 American Society of Civil Engineers (ASCE) Training Course "Fundamentals of Earthquake Engineering")







Ductility and Energy Dissipation Capacity

- The structure should be able to sustain several cycles of inelastic deformation without significant loss of strength.
- Some loss of stiffness is inevitable, but excessive stiffness loss can lead to collapse.
- The more energy dissipated per cycle, without excessive deformation, the better the behavior of the structure.

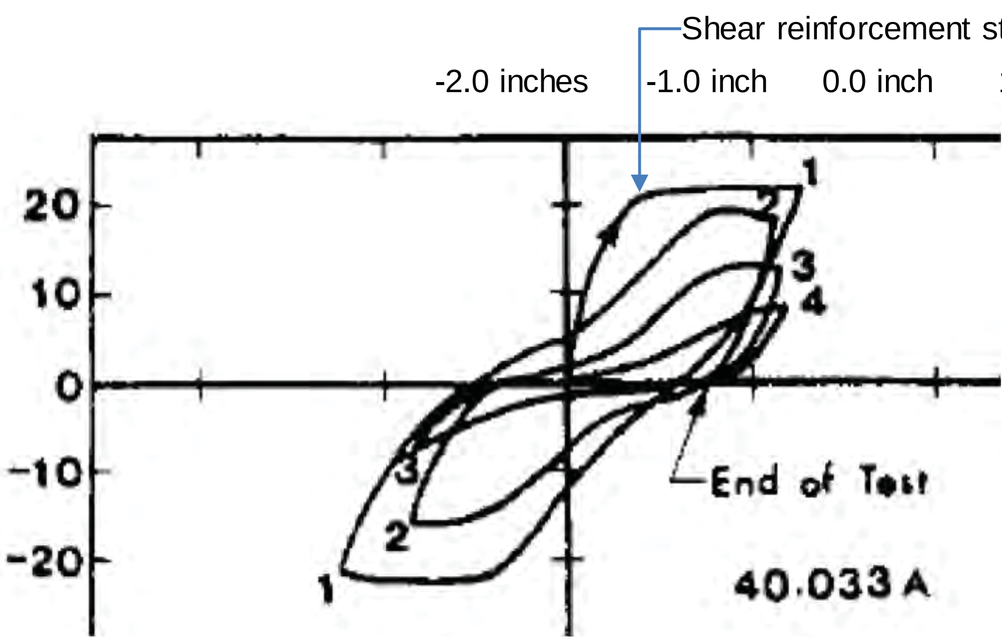
ASCE

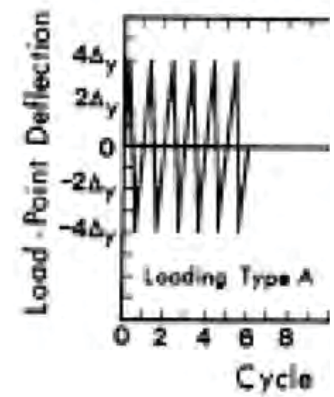
ACI Code's Energy Dissipation Capability (Toughness) Requirement

ACI 318-08 Code, Chapter 21, Earthquake-Resistant Structures, Section R21.1.1, states,

> "The design and detailing requirements should be compatible with the level of **energy dissipation (or toughness)** assumed in the computation of the design earthquake forces."

Hysteric Behavior of a Reinforced Concrete (RC) Element nearly Identical to Steel-Plate Concrete (SC) Wall Element in Shear



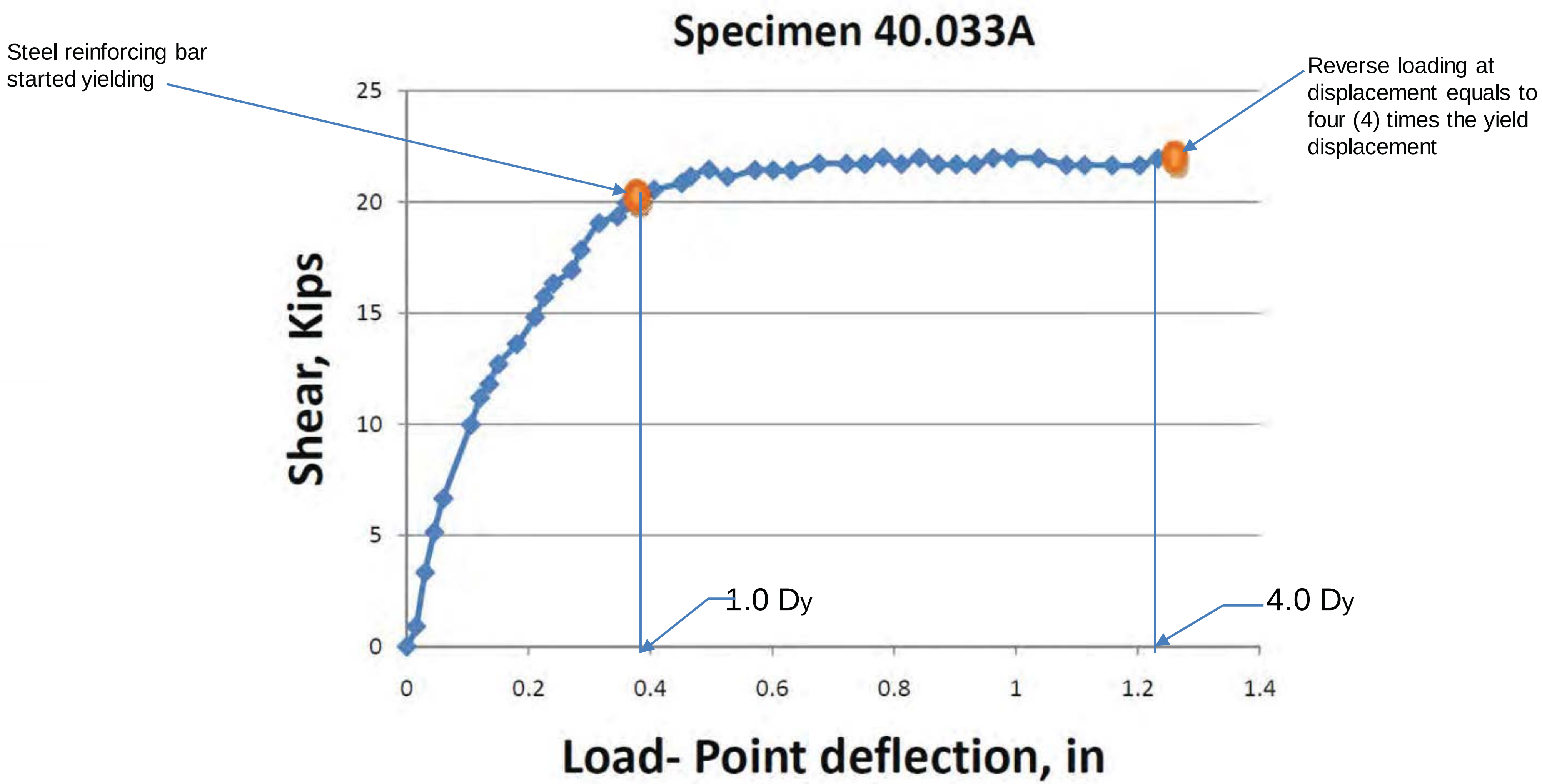


Load-point Deflection in inches

Excerpt from "Strength Decay of RC Columns Under Shear Reversals" by James K. Wight, and Mete Sozen, ASCE, Journal of the Structural Division, Vol.101, May 1975

RC Element First Quadrant Shear Load-Deflection Relationship

Tested with a displacement ductility ratio of four (4)



SC Wall Element Failed in the First Quadrant

a,c

Required Shear Ductility for AP1000 Steel-plate Concrete (SC) Wall Element

1. The NRC hired Dr. Hsu (Dr. Mo as alternate) of University of Houston and Dr. Vecchio of University of Toranto, two of the experts in concrete shell structural design and testing because their school own the only two testing machines in the world that could perform and had performed combined tests for out-of-plane and in-plane shears that shell structure would endure during earthquakes
2. Consistent with the ACI Code requirements, and after a week long face-to-face meeting between the NRC consultants and I, and the Westinghouse, the NRC and Westinghouse jointly established and agreed upon that the new SC wall element should possess a minimum displacement ductility (ratio) of [ ] a,c to be verified by both out-of-plane and in-plane shear tests. The logic is if the new structural wall element possesses a displacement ductility (ratio) of [ ] a,c in both orthogonal directions, wall elements would be sufficiently ductile to move in any of the 360 degree directions as they would during earthquakes without failure
3. Due to the new type of construction involved steel plates, the NRC also hired a consultant for steel welding, a consultant for construction, and a consultant for testing, but they were not involved in establishing the acceptance criteria for AP1000 shield building wall design because that was not their specialty and they had no experience in testing of combined out-of-plane and in-plane shears

Westinghouse Test Loading Protocol in Post-Yield Region

a,c

Comparison of Shear Strength and Ductility between SC & RC Elements

a,c

Comparison of Energy Absorption/Dissipation Capability between SC and RC Elements

a,c

95,000 psi Shear reinforcement Yield Strength in AP1000 Shield Building Wall Violates Code's limit of 60,000 psi

22.5.3.3 The values of f_y and f_{yt} used to calculate V_s shall not exceed the limits in 20.2.2.4.

R22.5.3.3 The upper limit of 60,000 psi on the value of f_y and f_{yt} used in design is intended to control diagonal crack width.

20.2.2.4 Types of nonprestressed bars and wires to be specified for particular structural applications shall be in accordance with Table 20.2.2.4a for deformed reinforcement and Table 20.2.2.4b for plain reinforcement.

R20.2.2.4 Tables 20.2.2.4a and b limit the maximum values of yield strength to be used in design calculations for nonprestressed deformed reinforcement and nonprestressed plain spiral reinforcement, respectively.

Excerpt from ACI 318-2014 Code

Note: Shear reinforcement in AP1000 shield building wall has a yield strength about [ ] a,c which violates the Code's limitation of 60,000 psi, and that contributed to brittle failure

Westinghouse's In-Plane Shear Test

1. NRC consultants commented that the test model was incorrect, because the [ ] a,c (similar to a picture frame takes the shear force instead of the thin picture inside of the frame is supposed to be tested and the picture would not fail even at high shear stress because it was bonded and restrained by the frame)
2. During a telephone conference between NRC consultant, Dr. Vecchio and Westinghouse, Dr. Vecchio told Westinghouse that the test set-up was incorrect, but Westinghouse insisted that it was too late to change
3. The [ ] a,c

NRC Consultant Dr. Vecchio's Comment on the Brittle Wall Element Failure Test

1. "The test beams representing the Shield Building wall detail [ ] a,c tested in out-of-plane shear (OOPS) at Purdue, exhibited shear-critical brittle failures."
2. "The one Purdue specimen [ ] a,c tested under cyclic shear was done so with a span-to depth (a/d) condition of 2.5. This specimen showed adequate ductility. However, the monotonic out-of-plane shear tests preformed previously indicated that [ ] a,c I recommend that the test be repeated for [ ] a,c
3. "The intent of Chapter 21 of ACI-349 (Provisions for Seismic Design) is that a structure be designed to behave, at its ultimate limit state, in a ductile manner. This necessitates that the structure exhibit a failure mechanism that involves yielding of the principal reinforcement such that a high deformation capacity be achieved, providing sufficient energy dissipation and avoiding all brittle failure mechanisms."

NRC Consultant Dr. Hsu's Comment on the Brittle Wall Element Test

1. "The most sensible approach to design the nuclear containment structure is to recognize the uncertain nature of the earthquake demands, and to provide the ductility that could redistribute the stresses so that all the reserve capacity could be mobilized to avoid a catastrophic failure. Hence, the ductility and the proper detailing of the structures to prevent brittle failure at possible weak locations are crucial."
2. "However, tests at Purdue University showed that the SC modules failed by out-of-plane shear in a brittle manner. The question before us is: Is the SC wall with [ ] a,c safe?"
3. "The stress in the cylindrical wall away from connection zones is actually under combined stresses of out-of-plane shear stress, in-plane shear stress, and axial stresses. Bending stress will also be added near the connection zones during earthquakes. In the opinion of the writers, a ductility ratio of [ ] a,c required by NRC is a reasonable one and needs to be implemented."

NRC/NRO/DE Management Decision on Ductility Issue

1. All other shear tests failed to meet the acceptance criteria jointly established and agreed upon by the NRC and Westinghouse except one shear test with the [

   ] a,c

   that met the acceptance criteria
2. Despite the oral and written protest from the NRC consultants: Dr. Vecchio[1], Dr. Hsu[2], and me[3], the NRC/NRO/DE management accepted the brittle wall element by ignoring the acceptance criteria and the ACI Code's requirement
3. As a result of the management decision, the majority of wall elements in AP1000 shield building wall are indeed **<u>brittle and unsafe</u>**

Footnotes:

1. "Ductility In Shear Response Of Shield Building," by Frank Vecchio, July 9, 2010
2. "Comments on the Safety of AP1000 Shield Building," by Thomas T.C. Hsu and Y.L. Mo, Nov. 14, 2010
3. "Memorandum of Understanding on the Resolution of Ductility Issue of SC Wall," by John S. Ma, June 13, 2010

Falsehood of DPO Panel on Wall Element Ductility Issue

1. Responding to a DPO panel member's May 3, 2013 9.38 AM email request on the source of the comparison curves of the RC vs. SC elements (see slides #29), I sent an email the same day on May 3, 2013 10.03 AM to the person and cc to other DPO panel members with the following information:
    1) The comparison of the SC and RC element curves were prepared by Dr. Mo.
    2) I requested Dr. Hus for an independent review for the "correctness and applicability" of the two curves
    3) Dr. Hsu reviewed the two curves and confirmed the "correctness and applicability" of the two curves
    4) The e-mails and the back-up calculations for the RC curve (slide #25) and SC curve (slide #26) were all enclosed in my e-mail to the DPO panel members
2. The DPO panel's February 24, 2014 report stated "The panel agrees with the submitter in that the ACI Code requires that ductility should be in every single structural component of the structure,"
3. However, with no request for clarification or discussion on the information that I sent to the DPO panel on May 3, 2013, the same February 24, 2014 report stated that the comparison of the RC vs. SC elements was "misleading", because both curves would have very likely produced similar (ductile) results had the axial compressive force been applied to the RC element also to SC element
4. I responded in my March 27, 2014 review report on the DPO panel's February 24, 2014 report that had an axial compression load been applied to the SC element during the test, the element 's ductility would further be reduced and not increased as stated by the DPO panel's report
5. The DPO panel, again with no request for clarification or discussion on my response, it stated in its October 30, 2014 report that the comparison of the RC and SC elements should never have been made, and the comparison is a classic example of an apples to oranges comparison
6. Twice the DPO panel would not discuss the merits or its disagreements of the responses provided to it, and simply made a conclusion that my responses are "misleading" and the test wall element (the red curve) should have been ductile and did not identify any mistakes in the back-up calculations that I sent to the panel on May 3, 2013
7. Fair-minded professionals would not have twice made such an illogical decision to not discuss the responses that I provided but simply insisted that Dr. Mo, Dr. Hsu, and I, who are specialists in concrete structures and testing and all had received awards from the ACI, had made wrong comparisons of test data

Falsehood of DPO Panel on Wall Element Ductility Issue (continued)

8. In a report, dated February 23, 2015, the DPO panel stated "The SC panel with wide tie rod spacing demonstrated ductile behavior with a ductility ratio of [ ] a,c when tested with a shear-span-to-beam-depth (a/d) ratio of [ ] a,c and that "Brittle behavior of SC panels is seen only for somewhat lower shear-span-to-beam-depth (a/d) ratios, which is also the case for conventional reinforced concrete"
9. The newly found ductility ratio of [ ] a,c test by the DPO panel was a flexural (bending) test for flexural failure, not a shear test for shear failure.
10. Even if the a/d= [ ] a,c test were assumed to be a shear test and shear failure with a ductility ratio of [ ] a,c fair-minded professionals would and should have conservatively taken the brittle test result for the a/d=[ ] a,c as the wall element property and behavior under combined forces of flexural (bending) and shear, because ignoring the lower test value and choose the higher test value as the wall element's property would be illogical, dangerous, and unsafe
11. In an effort to dismiss or ignore the shear test that failed in a brittle manner, the DPO panel advanced its own shear failure theory that shear failure is only applicable to lower a/d ratio (a/d = [ ] a,c is lower than [ ] a,c
12. **However, the DPO panel's argument is contradicted by the SC wall test data because**
    1) **the brittle wall failure was associated with the test of a higher a/d= [ ] a,c value, which is more brittle, than the lower a/d=2.5 test (see Dr. Vecchio's comment in slide # 33)**
    2) **The cyclic test data on the shear reinforcement (tie wires) spaced at [ ] a,c with a/d= [ ] a,c attended ductility ratio more than [ ] a,c that met the acceptance criteria (see Dr. Vecchio's comment in slide # 33)**

Falsehood of DPO Panel on Wall Element Ductility Issue (continued)

13. The DPO panel used the statement “which is also the case for conventional reinforced concrete” to enhance its shear failure theory and argument. However, that statement is incorrect because concrete research has not resolved the shear problem or failure mechanism between about a/d=2 and a/d=4, usually referred as “Kani’s shear Valley”. Therefore, several shear tests need to be performed in that regions in order to obtain the lowest shear value for design or evaluation (slide #33), as [

    ] a,c

14. The DPO panel’s attribution of flexural (bending) test results of flexural ductility as shear ductility, attempt to dismiss or ignore the brittle wall element shear test data by attacking the credibility of the RC and SC wall comparison curves, and inventing its own shear failure theory, and enhancing the credibility of its shear failure theory on the SC wall element tests with an incorrect and irresponsible statement that its theory was an extension from RC shear failure theory, are **false**.
15. That **falsehood** successfully misled the NRO Director into believing that the brittle wall element had a shear ductility ratio of [ ] a,c and dismissed the wall brittleness issue (see NRO Office Director’s February 27, 2015 letter)

Conclusion on Wall Ductility Test Results and NRO Management Decision

1. The majority of wall elements in AP1000 shield building wall are indeed brittle because the NRC/NRO/DE management accepted it in 2010 (see slide #26)
2. The ACI Code, which is the design/licensing basis of the AP1000 shield building, prohibits the use of brittle wall elements, as presented in slides #21 and #23)
3. The ASCE training course “Fundamentals of Earthquake Engineering” warns that the use of brittle wall elements is “unacceptable” (slide #22)
4. The DPO panel’s attribution and incorrect and irresponsible statements on concrete shear ductility issue are **false and unsafe** with respect to structural engineering principles on concrete structures and test results
5. However, these **falsehood** had successfully misled the NRO Director into believing that the wall elements are ductile and to dismiss the brittle wall element issue in his February 27, 2015 letter
6. The EDO is urged to see these **falsehood** and right the wrong of this fundamental structural design problem of AP1000 shield building wall that contains brittle structural wall elements which is unsafe and unprecedented in modern structural design practices

ACI Code's Shear Strength Requirement or Shear Stress Limitation for Wall Design

Shear Stress in Wall is Limited to **8$\sqrt{fc'}$**

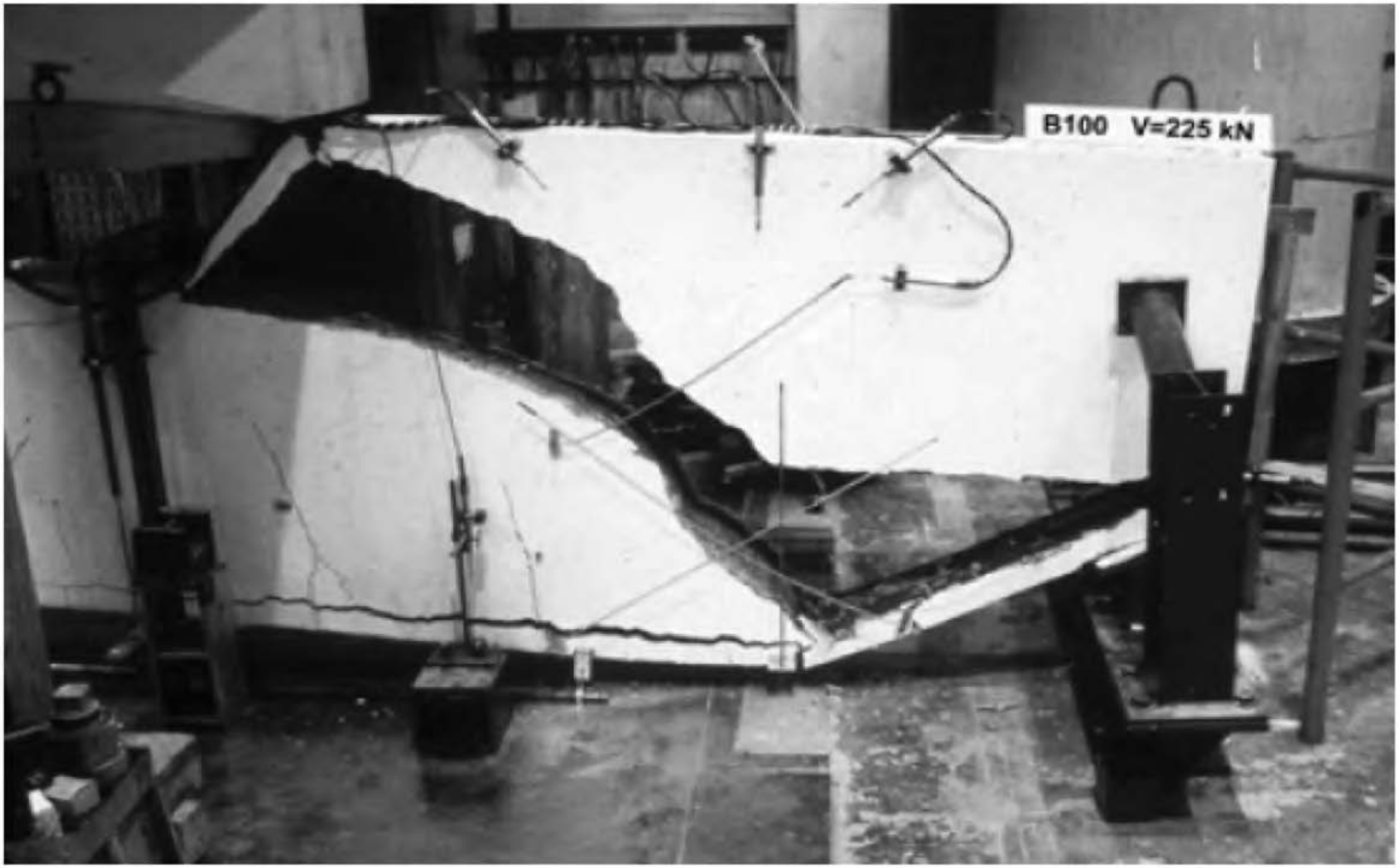
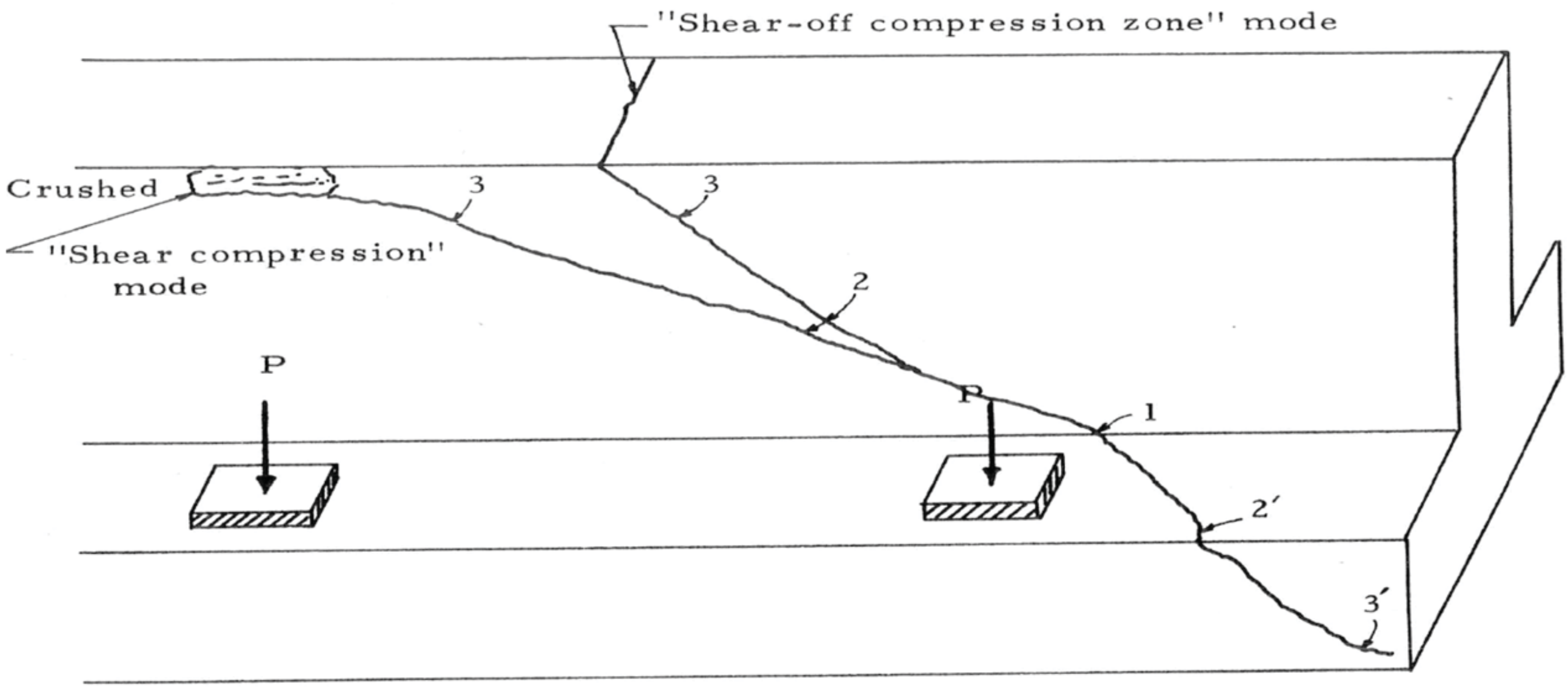


Fig. 1—Brittle shear failure of 39 in. (1000 mm) deep member without stirrups.[2]

"Where is shear Reinforcement required?" by Michael P. Collins, Evan C. Bentz, and Edward G. Sherwood, ACI Structural Journal/September-October 2008

Shear Compression and Shear-Off Failure Modes

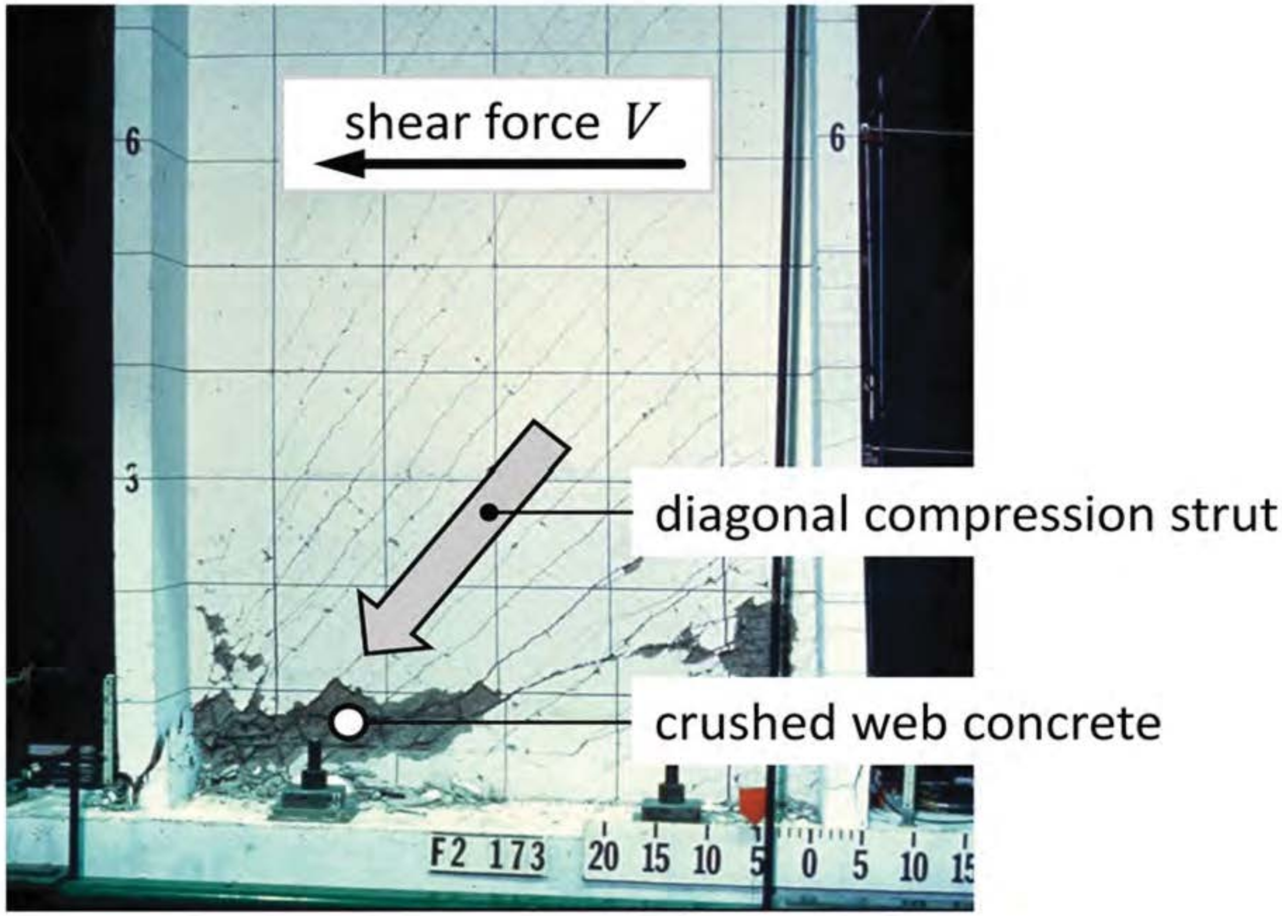


Shear compression failure mode and shear-off failure mode tested and recorded by John S. Ma

Fig. 3.14 Typical Web Failure Patterns

Excerpt from "Concrete Structure Fundamentals" Class material for NRC staff and taught by John S. Ma at PDC

Shear Caused Compression Failure of Concrete Wall Material



Web crushing due to high shear force in laboratory test

Excerpt from "Seismic Design of Cast-in-place Concrete Special Structural Walls and Coupling Beams", A guide for Practicing Engineers, by NEHRP and published by NIST, March 2012

Shear Plus Axial Compression Caused Wall Buckling

Wall buckling during 2011 earthquake in Christchurch New Zealand

Excerpt from “Seismic Design of Cast-in-place Concrete Special Structural Walls and Coupling Beams”, A guide for Practicing Engineers, by NEHRP and published by NIST, March 2012

Crushing, Spalling of Concrete and Buckling of Vertical Steel in walls due to High Compressive Force and Shear

during 2010 Chile Earthquake

Excerpt from “Seismic Design of Cast-in-place Concrete Special Structural Walls and Coupling Beams”, A guide for Practicing Engineers, by NEHRP and published by NIST, March 2012

High Compression and Shear Forces Caused Wall Steel Fracture and Buckling

during 2010 Chile Earthquake

"Damage and Implications for Seismic Design of RC Structural Wall Buildings", by John W. Wallace, Leonardo M. Massone, Patricio Bonelli, Jeff Dragovich, Rene Lagos, Carl Luders, and Jack Moehle, July 26, 2011

Three Basic Shear Failure Modes and Shear Strength Upper Limit

1. Shear Failure in Diagonal Tension Mode (slide #41)

   No, or very little, shear reinforcement (shear strength = **2√fc’** for no shear reinforcement)

2. Shear Failure in Compression Mode (slide #42)

   A lot of, or excessive, shear reinforcement, but concrete material is being crushed (ACI Code limits shear strength for walls = **8√fc’**)

3. Shear Failure in Sheared-off Mode (slide #42)

   A lot of, or excessive, shear reinforcement, but concrete material is being sheared-off and crack surfaces slide against each other (ACI Code limits shear strength for walls = **8√fc’**)

NOTE: ACI Code’s shear strength for walls is about **2√fc’** for no shear reinforcement up to **8√fc’** regardless of excessive shear reinforcement because concrete material could fail beyond that value and thus wall could collapse due to instability problem

Strength Requirement for Concrete Shell Design

ACI 318-2011, Section 19.2.7, states:

"The **thickness** of a shell and its reinforcement shall be proportioned for the **required strength** and serviceability,.."

Note: Concrete shear strength is linearly proportional to its wall thickness

ACI 318-2014 Code's Shear Strength Requirement for Walls Subjected to Earthquakes

R18.10.4 of ACI 318-2014 Code, Shear Strength, states:

"If the factored shear force at a given level in a structure is resisted by several walls or several vertical wall segments of a perforated wall, the average unit shear strength assumed for the total available cross-sectional area is limited to **8√fc'** with the additional requirement that the unit shear strength assigned to any single vertical wall segment does not exceed**10√fc"**

Forces Acting On A Wall

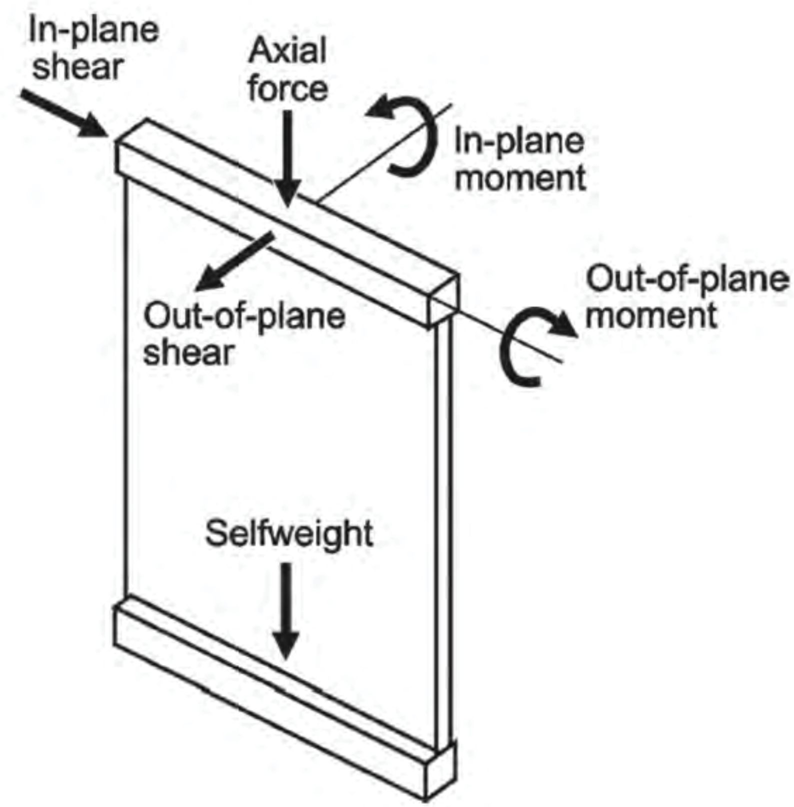


Fig. R11.4.1.3*—In-plane and out-of-plane forces.*

Excerpt from ACI 318 – 2014 Code Commentary

Walls Shall Be Designed for Maximum in-plane and out-of-plane Shears

11.4.3 *Factored shear*

11.4.3.1 Walls shall be designed for the maximum in-plane V_u and out-of-plane V_u.

Excerpt from ACI 318-2014 Code

Wall's Design Strength at all Sections (Elements) with Interaction Effect

CHAPTER 11—WALLS

11.5—Design strength

11.5.1 *General*

11.5.1.1 For each applicable factored load combination, design strength at all sections shall satisfy $\phi S_n \geq U$, including (a) through (c). Interaction between axial load and moment shall be considered.

(a) $\phi P_n \geq P_u$
(b) $\phi M_n \geq M_u$
(c) $\phi V_n \geq V_u$

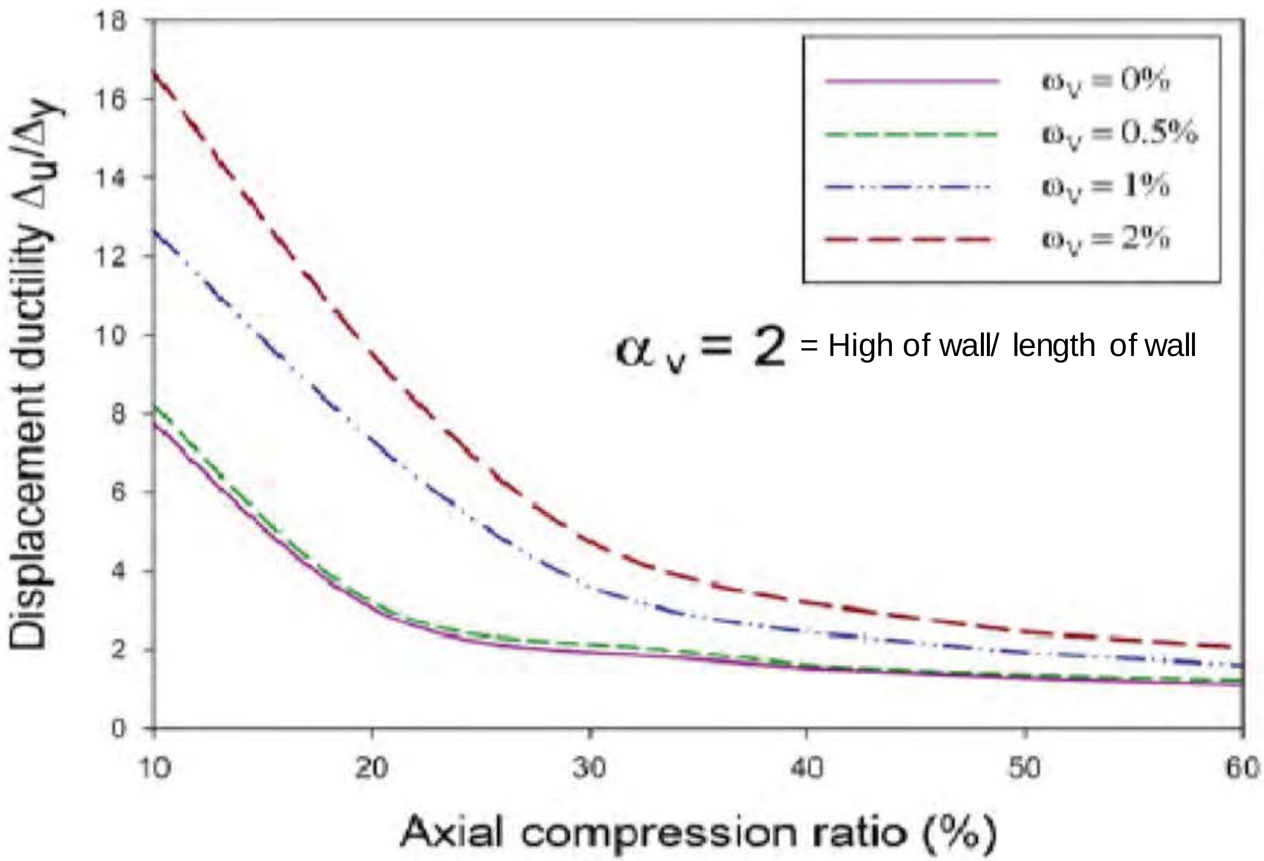
11.5.1.2 ϕ shall be determined in accordance with 21.2.

11.5.2 *Axial load and in-plane or out-of-plane flexure*

11.5.2.1 For bearing walls, P_n and M_n (in-plane or out-of-plane) shall be calculated in accordance with 22.4. Alterna-

Excerpt from ACI 318-2014 Code

Axial Load Reduces Wall's Flexural Ductility



= Applied axial load/axial load capacity of the cross-section

Excerpt from "Axial Compression Effect on Ductility design of RC Structural Walls", by Y.P. Yuen and J.S. Kuang. Second European Conference on Earthquake Engineering and Seismology, Istanbul, August 25-29, 2014

The Interaction Effect Between In-Plane and Out-of Plane Shears From Physical Tests

(the presence of the out-of-plane shear stress will reduce in-plane shear strength and vice versa)

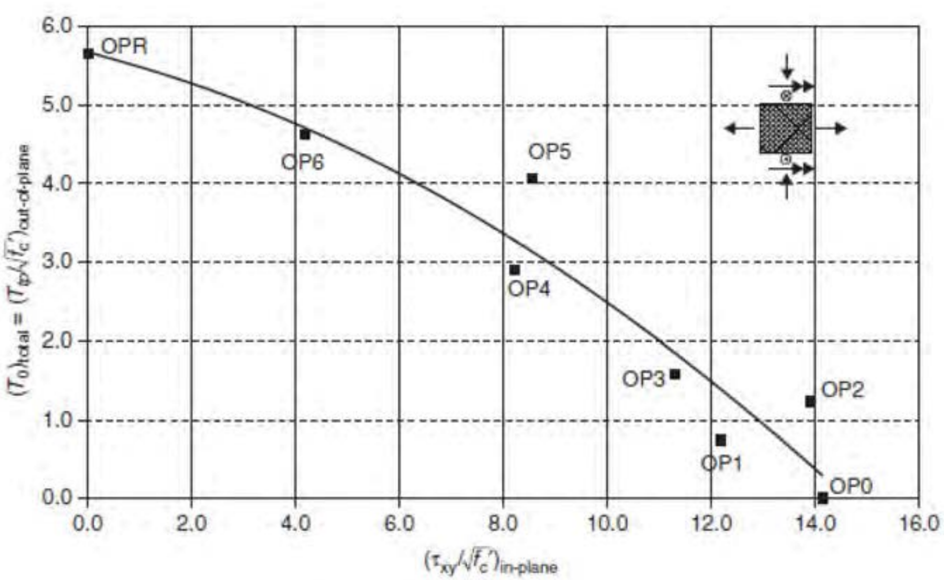
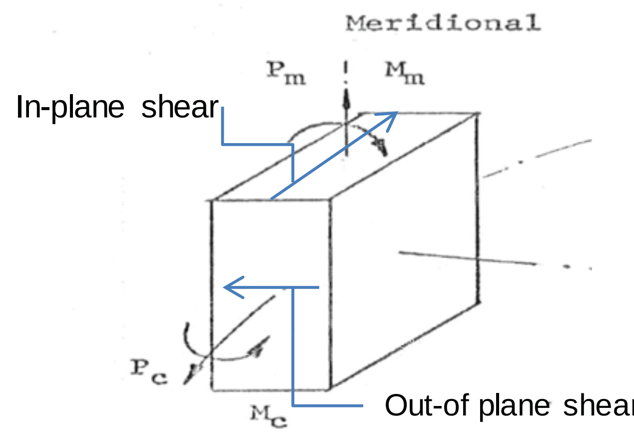


Figure 13.15 Interaction diagram between in-plane and out-of-plane shear strength



Shield building wall element

Excerpt from “Infrastructure systems for Nuclear Energy” by John Wiley and Sons, Ltd., Publication, 2014

__NOTE__: This interaction effect was not considered in the AP1000 shield building wall design and should have been done, as stated in one of the 130 questions in UK letter to the NRC, as shown in slide #.

Shear Reduces Flexural Ductility

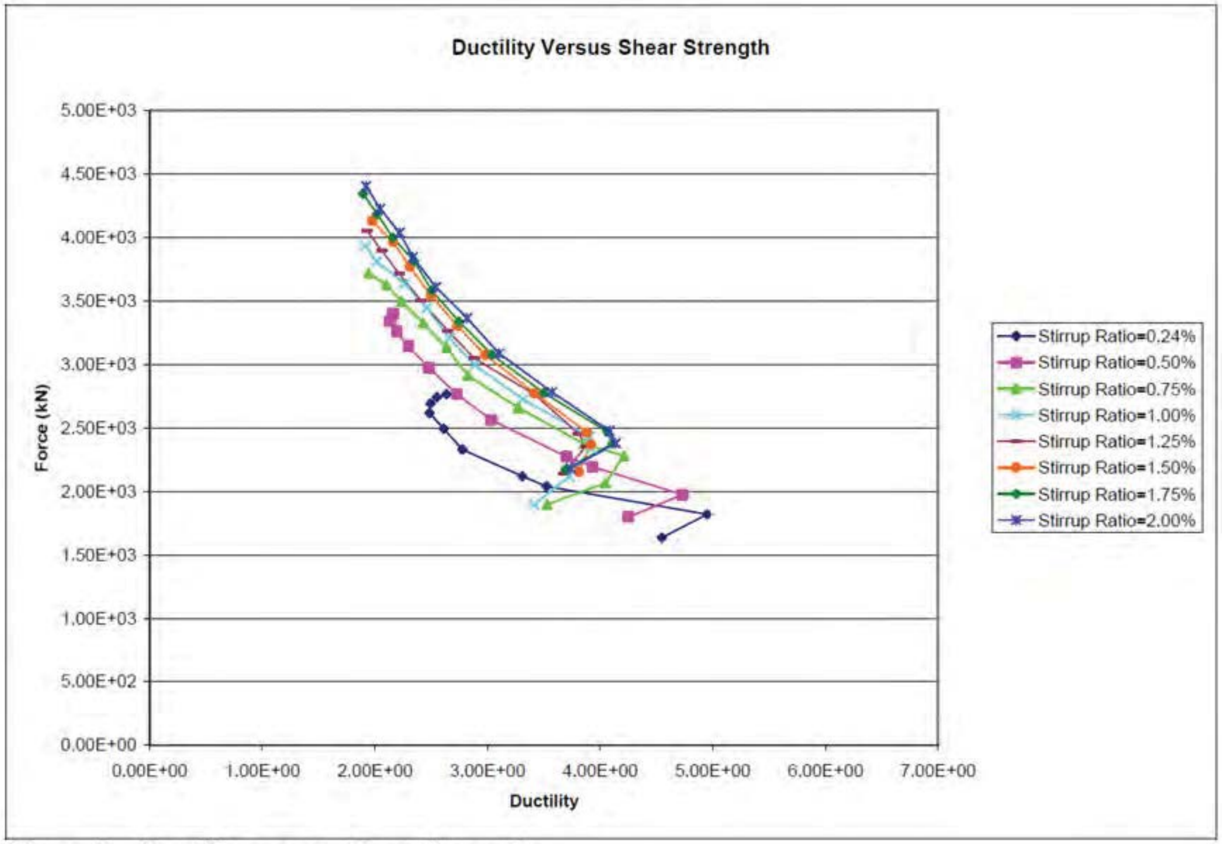


Figure 8—Ductility versus Shear Strength

"Seismic Interaction of Flexural Ductility and Shear Capacity in Normal Strength Concrete", by Rachel Howser, A. lascar, and Y.L. Mo, August 2007, Final Report, University of Houston, Houston, Texas, Sponsored by the National Science Foundation

ACI Code is Not a Recommended Practice, Complete Design Procedures, or Design Aids

ACI 318-2014 Building Code states:

"As the name implies, "Building Code Requirements for Structural Concrete" is meant to be used as part of a legally adopted building code and as such must differ in form and substance from documents that provide detailed specifications, recommended practice, complete design procedures, or design aids."

Design Guide Provides Specific Design Procedures

1. A **design guide** provides very specific guidance and design procedures for designing a specific type of structural members, such as a structural (shear) wall to resist earthquakes.
2. The following seismic design guide for concrete walls states: “The National Earthquake Hazards Reduction Program (NEHRP) Technical Briefs are published by the National Institute of Standards and Technology (NIST), as aids to the efficient transfer of NEHRP and other research into practice, thereby helping to reduce the nation’s losses from earthquakes.”

NIST GCR 11-917-11REV-1

NEHRP Seismic Design Technical Brief No. 6

Seismic Design of Cast-in-Place Concrete Special Structural Walls and Coupling Beams

A Guide for Practicing Engineers

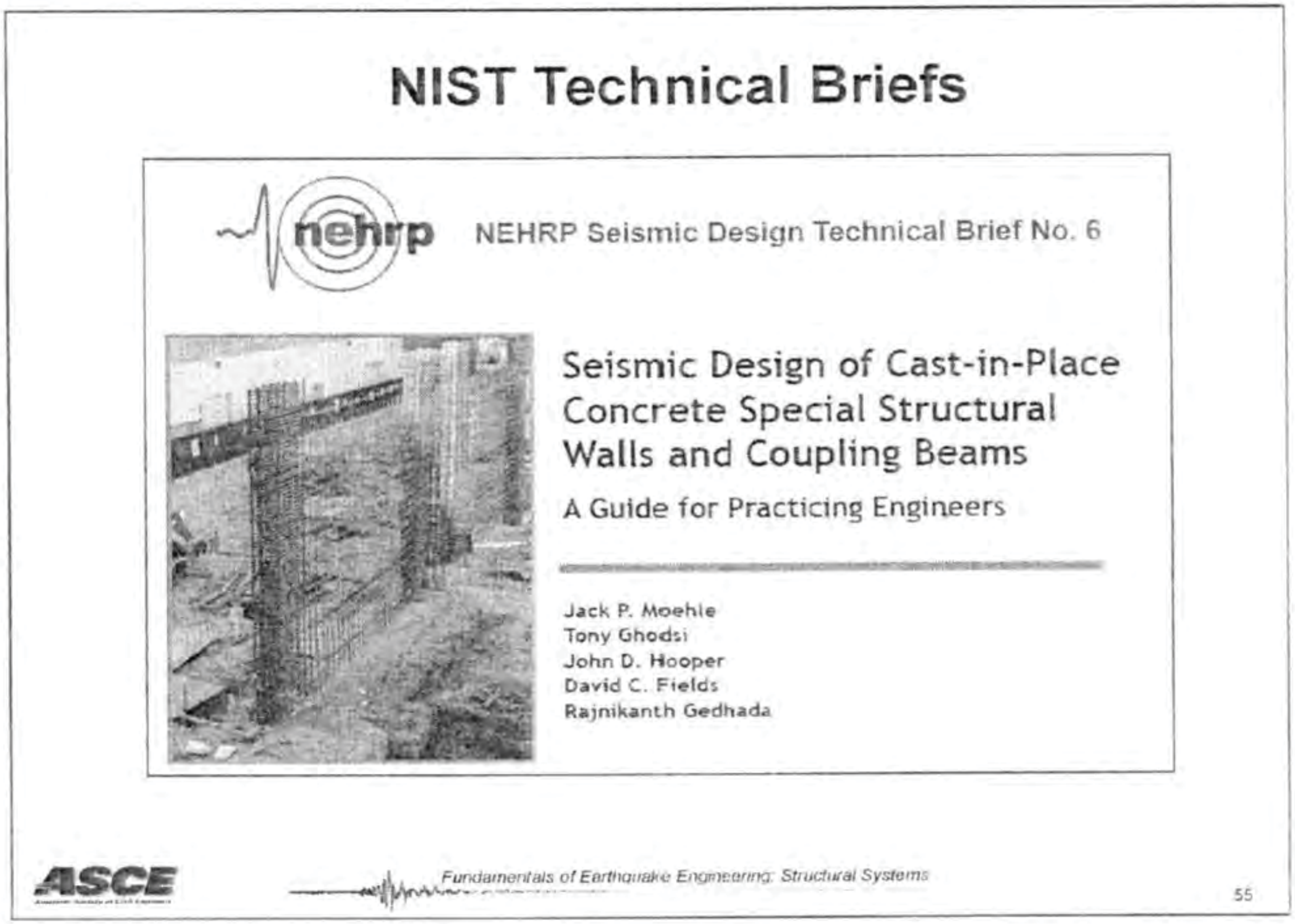
Jack P. Moehle
Tony Ghodsi
John D. Hooper
David C. Fields
Rajnikanth Gedhada

National Institute of Standards and Technology
U.S. Department of Commerce

The National Earthquake Hazards Reduction Program (NEHRP) Technical Briefs are published by the National Institute of Standards and Technology (NIST), as aids to the efficient transfer of NEHRP and other research into practice, thereby helping to reduce the nation's losses from earthquakes. The partners in the NEHRP Consultants Joint Venture are the Applied Technology Council (ATC) and the Consortium of Universities for Research in Earthquake Engineering (CUREE).

ASCE 2016 Training Course Referred the same Design Guide for Concrete Wall Design

(This is the design guide for concrete walls in the structural professional design community)



Shear Stress of **4√f’c to 6√f’c** is Recommended for Wall Design

The NEHRP Design Guide States:

“Although ACI 318 permits factored shear on individual wall segments as high as *Vu* = 10ϕ√*f’c* *Acv*, the flexural ductility capacity for such walls is reduced compared with identical walls having lower shear. This Guide recommends factored shear, calculated considering flexural overstrength (see Section 3.1.3), not exceed approximately **4ϕ√f’c Acv to 6ϕ√f’c Acv** so that **flexural ductility capacity** is not overly compromised.”

Proper Shear Stress Value for AP1000 Shield Building Wall Design

1. The NEHRP design guide provides a range of values from **4√f’c** to **6√f’c** for wall design depending on the complexity ands severity of effects due to stress combinations
2. The AP1000 shield building wall has high shear stress due to the PCS tank (slide #14) and the irregular anchorage (slides #14 and #15) during earthquakes, higher compressive stress than existing or other new shield building walls due to the weight of the PCS tank that would reduce wall flexural ductility (slide #53), and has interaction effect of in-plane and out-of-plane shears (slide #54) during earthquakes that would reduce both in-plane and out-of-plane shear strengths (slide #54) and flexural ductility (slide #55)
3. The stress in the wall is a combined stress of out-of-plane shear stress, in-plane shear stress, and axial stresses (slide #34). Therefore, the selection of the lower end value of **4√f’c** should be used for design in conjunction with the SSE loading

SSE Shear Significantly Exceeds Design Shear $4\phi\sqrt{f_c}$

RLE Shear significantly Exceeds Code's Limit $8\phi\sqrt{f_c}$

a,c

AP1000 Shield Building Wall Thickness is Too Thin

Since shear stresses over the height of the wall (SSE curve) exceed the NEHRP recommended design shear strength of **4√f'c,** as shown in the above slide, the wall thickness is under-designed and needs to be revised

Falsehood of DPO Panel on Wall Shear Strength Design

1. With regard to the Westinghouse's use of [ ] [a,c] as wall shear strength, which exceeded the ACI Code's limit of **8√f'**, the DPO panel picked its consultant's opinion of **[ ]** [a,c] over the Code's limit, but was silent on the fact that the same consultant also endorsed the NEHRP design guide that recommends design shear strength values from **4√f'c** to **6√f'c**.
2. Choosing an individual's opinion over the ACI Code's limit that is the design/licensing basis of the AP1000 shield building is legally wrong. Ignoring the fact that the same individual also recommended the NEHRP design guide that should have resulted in about [ ] [a,c] Westinghouse had used is morally wrong
3. While the DPO panel tried to defend Westinghouse's use of [**]** [a,c] as proper wall shear strength, it should have addressed the issue of significant exceedance of RLE shear stress over the **[ ]** [a,c] as stated in my DPO, dated June 29, 2012, and is shown on slide #62

Falsehood of DPO Panel on Wall Shear Strength Design (continued)

4. The DPO panel did not, or was unwilling to, answer my question on its view on the ACI Code's shear strength limit of **8√f'c** or the NEHRP design guide's recommended values from **4√f'c** to **6√f'c**
5. Furthermore, the DPO panel did not, or was unwilling to, answer my question on the √f'c values it considered to be adequate for the evaluation of shear stresses generated by SSE or RLE.
6. Without answering the question of shear strength or stress limiting values, it is not possible to make a determination whether a design is adequate or not because there is no reference point, but can only serve the purpose for the DPO panel to state that the AP1000 shield building could sustain RLE without being challenged because there is no reference point to be compared with

Falsehood of DPO Panel on Wall Shear Strength Design (continued)

7. The Shear strength or stress limiting criteria in the ACI Code and the NEHRP design guide for concrete walls were derived from test data and analysis results of wall collapses or failures resulting from earthquakes that reflect the **true** failure criteria for concrete walls
8. There are defined criteria and procedures in the ACI Code and the NEHRP design guide for concrete wall design and evaluation, which are used by practicing structural engineers
9. However, the DPO panel totally disregarded these defined criteria and procedures in the ACI Code and the NEHRP design guide for concrete walls, and provided **no criteria and no consistent procedures,** except cherry picking tactic, on how it had used to evaluate the wall adequacy
10. Unwilling to provide design/evaluation criteria, the DPO panel’s statement of adequate wall design is baseless and **false**

Conclusion on Inadequate Shear Strength Design

1. The Maximum shear stress generated by the RLE exceeded the ACI Code's limit [ ] a,c
2. The Maximum shear stress generated by the SSE exceeded the NEHR design guide's shear stress also [ ] a,c
3. The deficiency of design shear strength [ ] a,c is an evidence that the design is **unsafe**
4. The DPO panel's cherry picking tactic and unwilling to face or address the ACI Code's shear strength (capacity) that is the design/licensing basis for AP1000 shield building and the NEHRP design guide procedure for concrete wall design that the structural design professional community uses are improper
5. Unwilling to face the ACI Code and the NEHRP design guide or provide its own shear stress limits for wall design, the DPO panel's statement of wall adequacy is baseless, disingenuous, and **false**
6. The EDO is urged to see this **falsehood** and right the wrong

Seismic Margin Analysis

UK Nuclear Regulator Did Not Believe AP1000 Shield Building Could Meet NRC's SMA
(UK provided 130 questions on AP 1000 shield building design to the NRC on October 10, 2010)

1. "Explain how the NRC expectation of seismic margin for 1.67 x SSE shaking is met." – Question 75

Implication: UK did not see that NRC Seismic margin requirement could be met

2. "Consideration to combined in- and out-of-plane shear should be given in the development of the acceptance criteria." – Question 60

Implication: UK saw no shear interaction was considered in design that should have been considered (slide #54)

3. "The statement that the Shield Building is ...designed such that it exceeds the strength and ductility of RC structures" makes no sense."- Question 90

Implication: UK concluded that WEC's statement had no basis and was irresponsible

UK's belief on Wall Not Being Able to Meet NRC's SMA Requirement is Obvious because RLE Shear Stresses Exceed $8\sqrt{f_c'}$ Significantly

(wall stability problem resulting from concrete material being crushed or sheared-off)

a,c

Reasons for Not Being Able to Meet SMA

1. The PCS tank is heavy and placed on top of the shield building – the highest and worst location that generates the maximum seismic shear in the building wall, and this kind of high seismic shear stress is unique to AP1000 shield building because other shield buildings do not have a PCS tank and would not have this kind of high seismic shear stresses
2. The majority of wall elements are brittle and could not redistribute shear stress that exceeds its shear strength to other regions with less shear stresses (slides #26 and #34)
3. The Maximum shear stress generated by the RLE exceeded the ACI Code's shear strength limit [ ] a,c which means concrete materials in the wall are likely to be crushed or sheared-off - an indication of insufficient wall thickness
4. The wall of the AP1000 shield building is the sole structural system that resists both vertical load (supporting the PCS tank) and horizontal loads generated by earthquakes or winds (tornado or hurricane), and concrete material failures (crushed or sheared-off) could cause wall collapse and create unintended early and large release of radiation into the environment

Aircraft Impact Analysis

ANSYS

2006 International ANSYS Conference

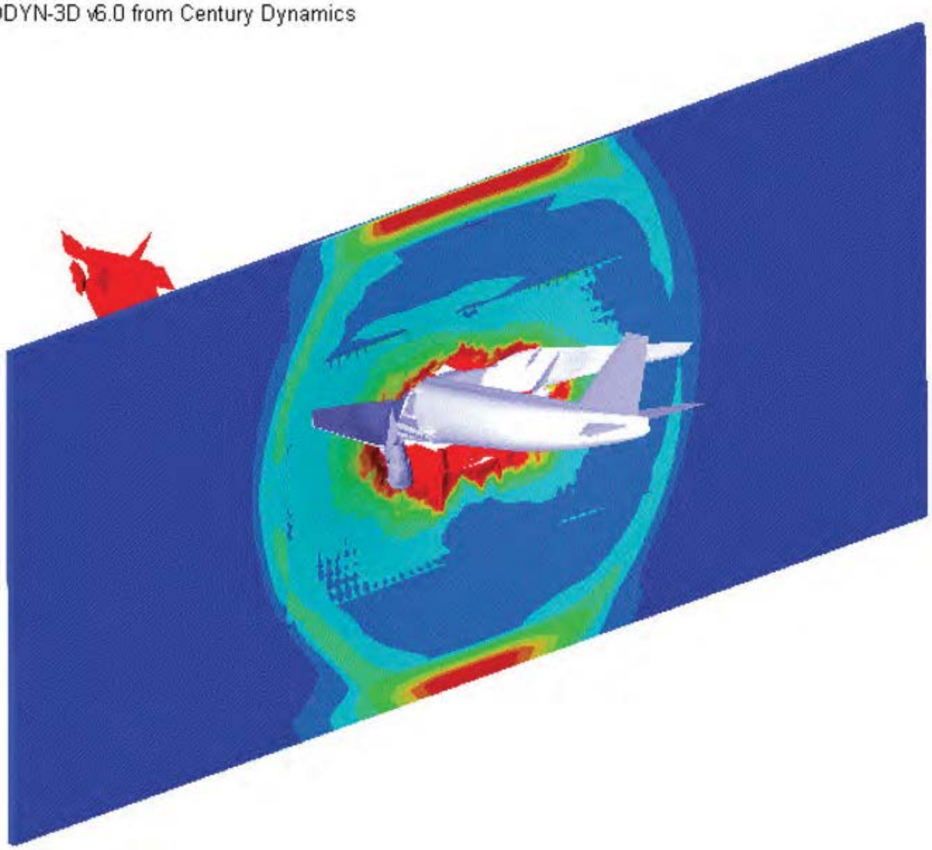
May 2-4, 2006

Damage of Reinforced Concrete Walls from Shock and Impact

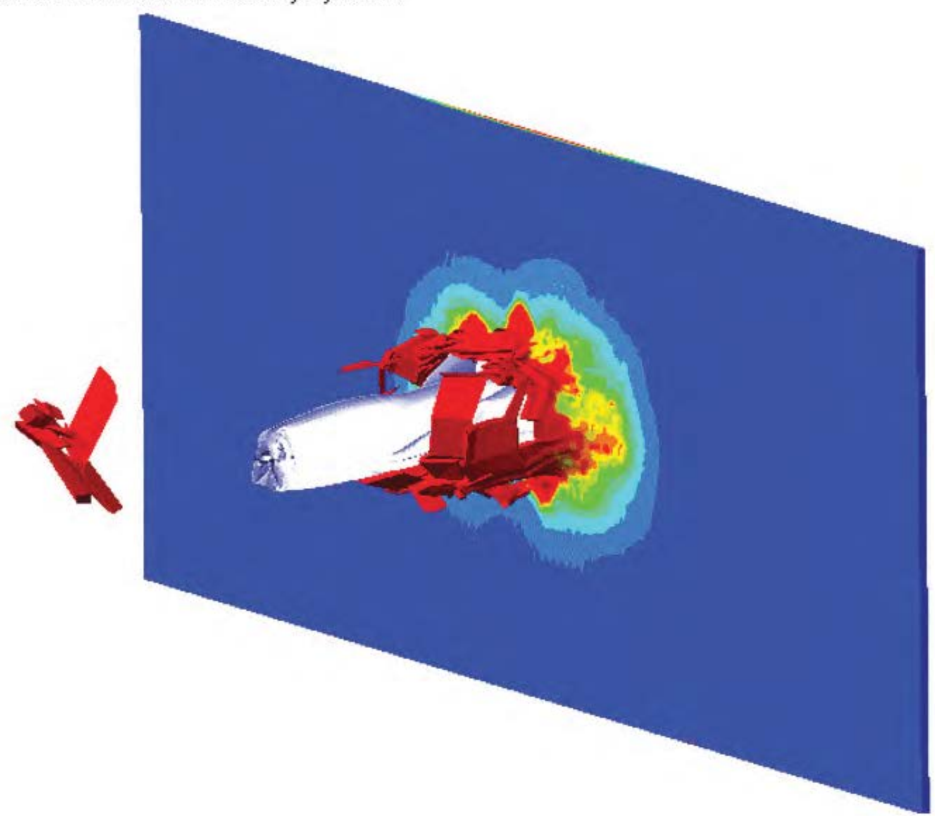
Coupled Multi-Solver Approach

X. Quan
Development Engineer
Century Dynamics, ANSYS, Inc.

Damage: 1m Thick

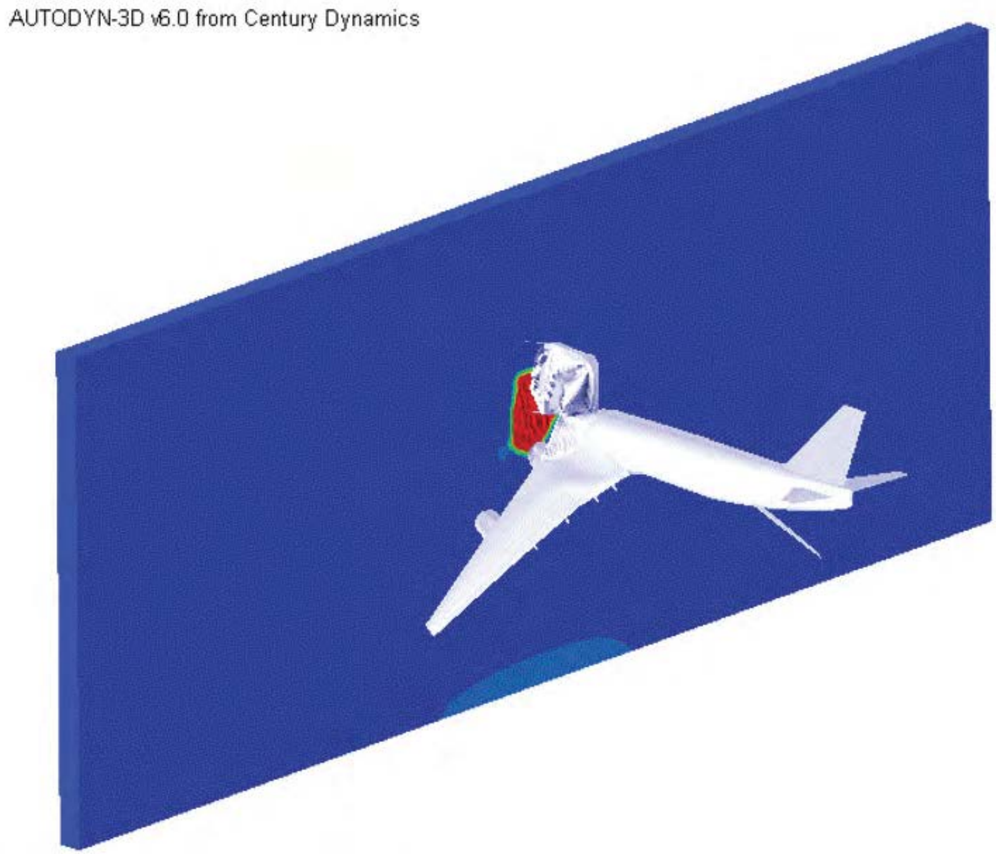


Front View

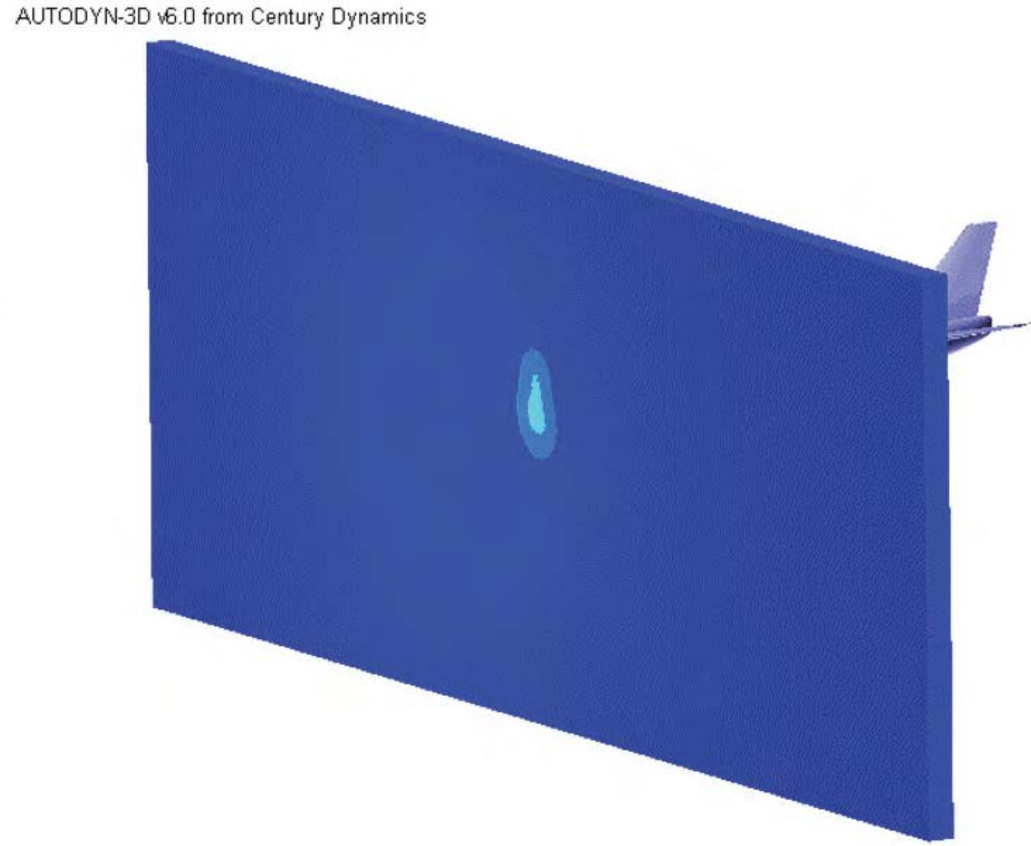


Back View

Damage: 3m Thick



Front View



Back View

Summary

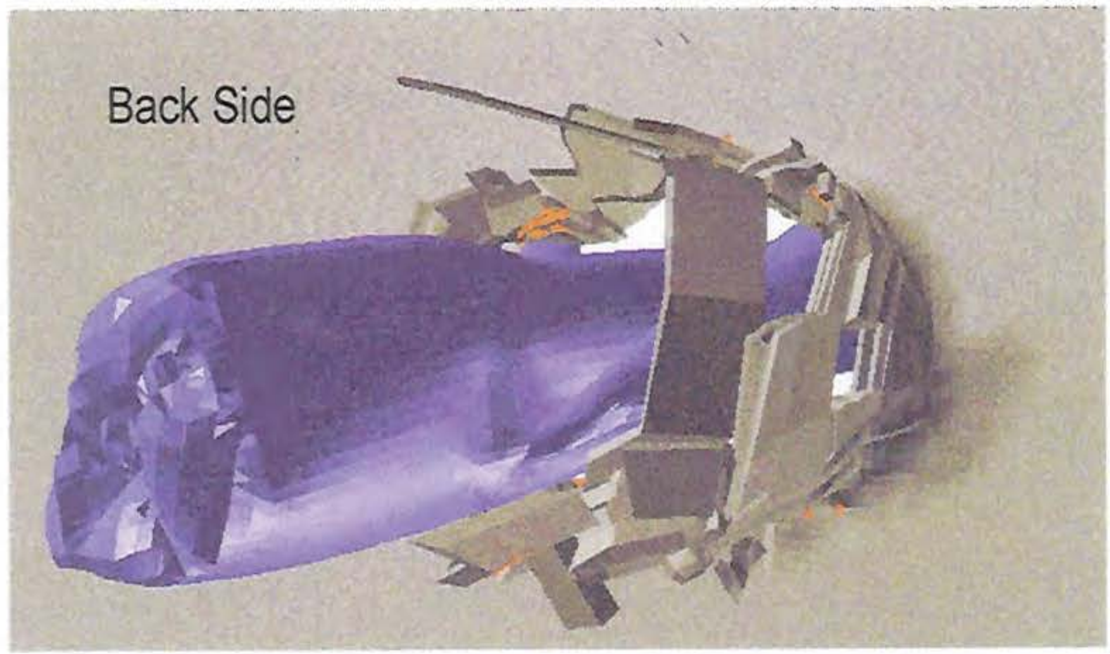
Summary

- 1m & 3m thick, 150m wide, 60m high steel-reinforced concrete walls are considered
- Walls are impacted by a Boeing 747 passenger jet
- Impact velocity: 83.3m/s (300km/s)
- 1m thick wall fails under the impact
- 3m thick wall withstands the impact

Aircraft Impact Analysis

Perforation of 1m (3.28 feet) RC Wall

Note: The failure mode is a punching shear failure of approximately 45 degree cone shape, not a bending failure as assumed by the DPO panel.

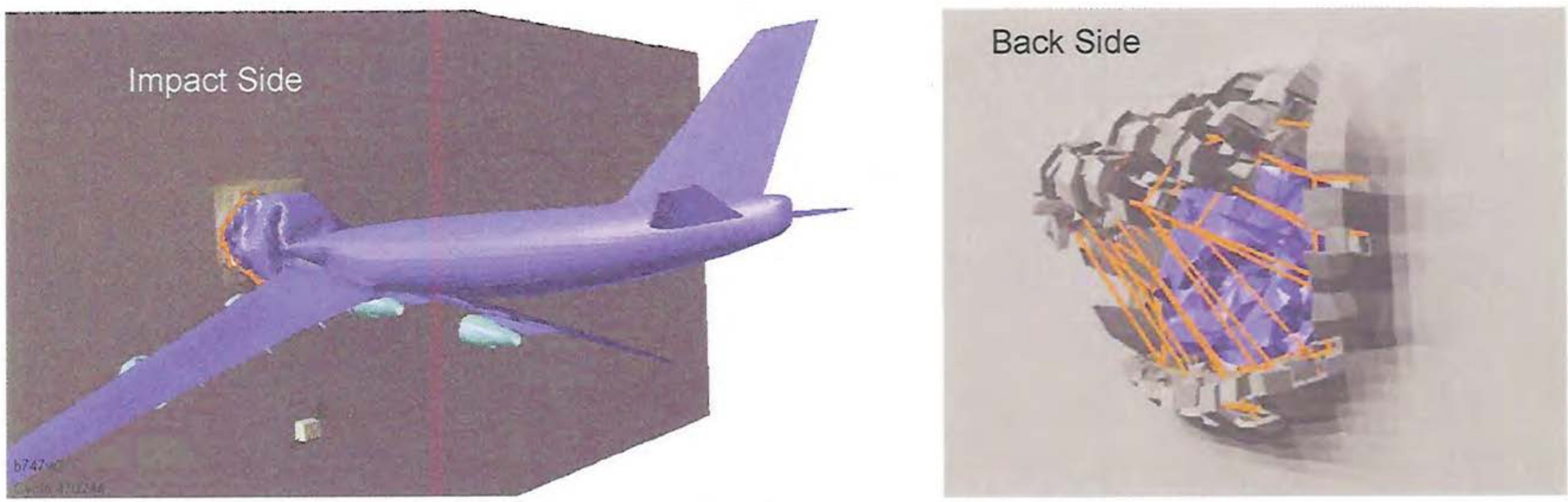


(a) CASE-1 (Thickness: 1 m, with RF), Time: 1 s.

Reference: “Numerical Simulation of Jumbo Jet Impacting on Thick Concrete Walls – Effects of Reinforcement and Wall Thickness” by Masahide Katayama, Masaharu Itoh, and Robert Rainsberger, Presented at the Second Asian Conference on High Pressure Research (ACHPR-2), in Nara, Japan on Number 1-5, 2004

Aircraft Impact Analysis

Rebars Stop Perforation of 2m (6.56 feet)Thick Wall



(b) CASE-2 (Thickness: 2 m, with RF), Time: 1 s.

Reference: “Numerical Simulation of Jumbo Jet Impacting on Thick Concrete Walls – Effects of Reinforcement and Wall Thickness” by Masahide Katayama, Masaharu Itoh, and Robert Rainsberger, Presented at the Second Asian Conference on High Pressure Research (ACHPR-2), in Nara, Japan on Number 1-5, 2004

Required Concrete Wall thickness for Aircraft Impact from 9 Analyses

Note: Except AP1000 shield building concrete wall thickness of 3 feet, other aircraft missile analyses require about 6 feet concrete wall thickness.

Type of Airplane	Type of design or plant name	Country performed the analysis	Weight of airplane	Velocity of impact	Computer code used	Required concrete wall thickness	Punch thru concrete wall thickness	Reference (See next slide)
Boeing 747		Italy	350,000 Kg (159,000 lbs)		MSC	1.6 meters (5.25 feet)	1 meter (3.28 feet)	1
MRCA		UK		215 meter/sec. (481 mile/hr.)	PISCES	2 meters (6.42 feet)	1 meter (3.28 feet)	2
Phantom RF-4E		Netherlands		215 meter/sec. (481 mile/hr.)	PISCES	1.8 meters (5.91 feet)		3
	Convoi	German				1.8 meters (5.91 feet)		4
	EPR	France German				1.8 meters (5.91 feet)		4
	ATMEA1	France				1.8 meters (5.91 feet)		4
	EC6	Canada				1.8 meters (5.91 feet)		4
	ESBWR	USA			ANATECH-ANACAP		1.07 meters (3.5 feet)	5
	South Texas	USA			ANATECH-ANACAP		1.22 meters (4.0 feet)	6
	SC wall module almost identical to that of AP1000 shield building	USA			LS-DYNA	1.68 meters (5.5 feet)	1.22 meters (4.0 feet)	7
	AP1000	USA			Westinghouse	0.91 meters (3.0 feet)		8
Boeing 747		Japan	340000kg	300 km/h	AUTODYN-3D	Greater than 2m	2 meters (6.56 feet)	9

Reference:

1. R. Lo Frano and G. Forasassi, “Preliminary Evaluation of Aircraft Impact on a Near Term Nuclear Power Plant,” Proceedings of the Intl Conference Nuclear Energy for New Europe, Sept. 14-17, 2009.
2. M.A.H.G.Alderson, eta, “Reinforced Concrete Behavior Due To Missile Impact,” 4th SMIRT Conference, Report J 7/7, San Francisco, USA, 1977.
3. Ree, H. v.d.,Hoek. M.J. v.d., ”Analysis of the Behaviour of a Containment Structure due to an Airplane Impact and the Effects of the Reinforcements,” 5th SMIRT Conference, Report J 9/5. Berlin, Germany, 1976.
4. CNSC follow-up questions to the NRC on the Apparent Insufficient Thickness of the AP1000 Shield Building Wall, March 2013.
5. GE Hitachi Nuclear Energy to NRC, “Reply to Notice of Violation, NRC Inspection report 05200010/2010-201.
6. South Texas Aircraft Impact Analysis Inspected by the NRC Staff.
7. T.R.S. Mullapudi, Paul Summers, and II-Hwan Moon, “Impact Analysis of Steel Plated Concrete Wall,” Structures Congress 2012 @ ASCE 2012.
8. Westinghouse Design Report for the AP1000 Enhanced Shield Building.
9. “Numerical Simulation of Jumbo Jet Impacting on Thick Concrete walls—Effects of Reinforcement and Wall Thickness” by Masahide Katayama, Masaharu, Koto-Itoh, and Robert Rainsberger, Presented at the Second Asian Conference on High Pressure Research (ACHPR-2), in Nara, Japan on November 1-5, 2004

Canadian Nuclear Regulator (CNSC) Questioned Adequacy of NRC Review Method and Conclusion on AP1000 Shield Building Design

1. "It was mentioned several times during two teleconferences that the code for reinforced concrete safety related nuclear structures ACI349 was used in a conservative way for the design of SC structures. How was defined an equivalent reinforced concrete structure for a given SC structure to make this statement?"

Implication: CNSC believed that NRC's statement was incorrect and had no basis (this NRC statement was borrowed from WEC which was judged by the UK regulator as "make no sense" in slide #68)

2. "One of the conclusions of the Mizuno's paper was that the use of SC structures can reduce the thickness by approximately 30%, comparing to conventional reinforced concrete structures. The thickness of conventional reinforced concrete shielding to protect against aircraft crash is 1.8 m (latest German Convoi NPPs, EPR, ATMEA1, EC6). The thickness of AP1000 shielding is 0. 9 m or 50% of the conventional reinforced concrete shielding. What is the rationale?"

implication: CNSC did not believe NRC's use of Mizuno's paper could result in that AP1000 shield building wall was thick enough

3. "Did US NRC perform any check analysis to confirm WEC finding?"

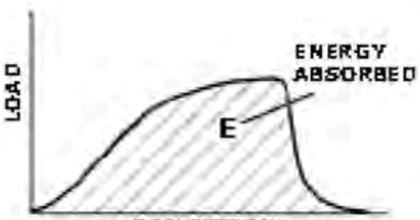
Implication: CNSC believed that NRC simply took WEC's finding as its own finding without checking

ACI Code's Ductility (Energy Absorption Capability) Requirement for Missile Impact

ACI 349-06, "Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary," Appendix RF – Special Provision For Impulsive and Impactive effects, Section RF.4 - Requirements to assure ductility, states

> "The provisions **to assure ductility** are parallel to appropriate sections of Chapter 21 of ACI 318-05."

Importance of Ductility for Missile Barrier Design

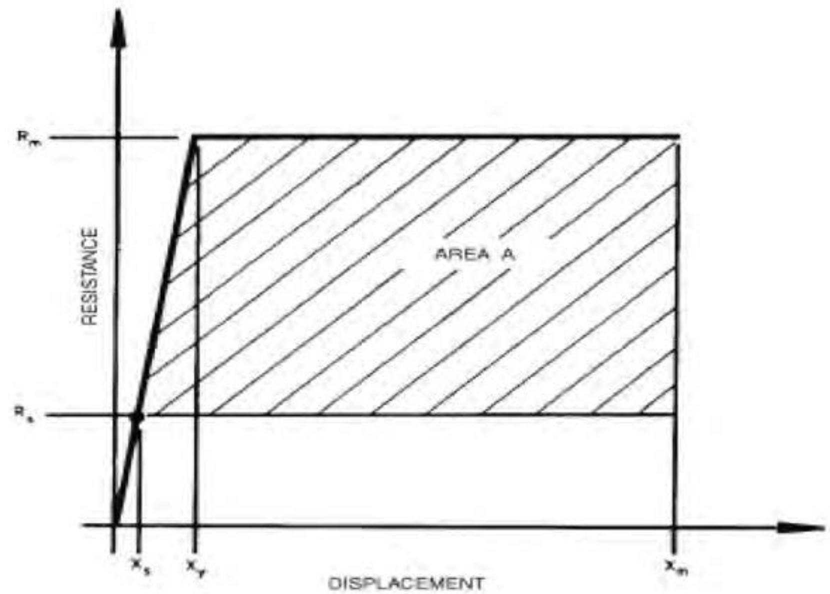


IMPACT ENERGY:
1) IMPACT ENERGY = POTENTIAL ENERGY + KINETIC ENERGY

ABSORBED ENERGY:
1) A MEASURE OF MATERIAL STRENGTH AND DUCTILITY
2) GRAPHICALLY THE AREA BENEATH THE LOAD DISPLACEMENT CURVE

“Brittle materials take little energy to start a crack, little more to propagate it to a shattering climax. Other materials possess ductility to varying degrees. Highly ductile materials fail by puncture in drop weight testing and require a high energy load to initiate and propagate the crack.”

(www.instron.com/wa/applications/**test**_types/**impact**/default.aspx)



$\mu_d = \frac{X_m}{X_y}$

R_s = STATIC FORCE TO BE COMBINED WITH IMPULSIVE OR IMPACTIVE LOADS

X_s = DISPLACEMENT DUE TO STATIC LOADS

Fig. RF.8—Available resistance: idealized resistance-displacement curve.

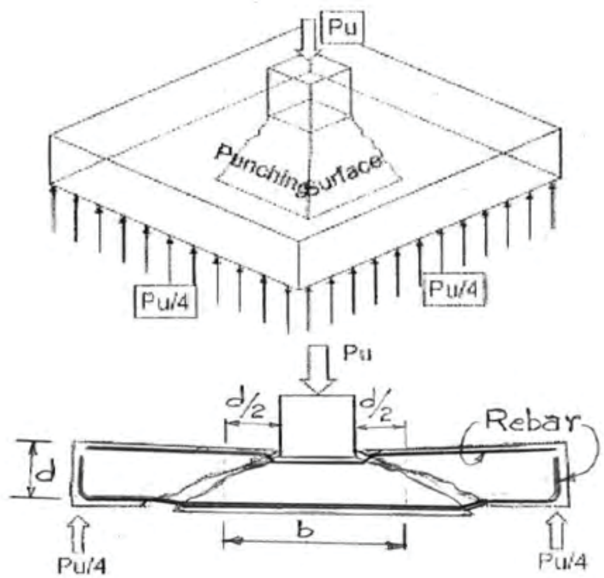
“If the energy balance method is used, only the energy represented by area A in Fig.RF.8, which is available to resist the impulsive and impactive loads, should be used.”

Excerpt from ACI 349-06 Code

Comparison of Ductility and Energy Absorption capability between SC and RC Elements

a,c

Punching Shear Failure Mechanism and Strength (Capacity) Equation



- Punching shear failure results in a pyramid or truncated cone of a slab or wall separated from the original slab or wall.
- The punching ultimate shear strength, based on the ACI Code, is Pu (ultimate) = $4\sqrt{f'c}\, bd$, where
  - f'c is concrete compressive strength
  - d is the effective depth of the slab or wall
  - b is the critical perimeter measured at ½ d from the column (punch) face as shown in the figure

NOTE:

*The ACI punching shear strength equation reveals that the punching shear strength is to the **square power of the thickness** of walls, slabs, and footings (a six feet thick wall has four (4) times punching shear strength of that of a three feet thick wall).*

Excerpt from "Concrete Structure Fundamentals" Class material for NRC staff and taught by John S. Ma at PDC

Falsehood of DPO Panel's Evaluation on AIA

1. The DPO panel stated that both Westinghouse and the NRC/RES staff had performed AP1000 shield building AIA by using the same computer code, and its consultant had modeled a portion of the wall as an arch with unyielding supports for analysis
2. The DPO panel stated that these three analyses results indicated no perforation and the failure mode was [ ] a,c and not punching shear
3. The wall element's flexural (bending) test yielded a [ ] a,c and the same wall element' shear test yielded in a brittle manner with [ ] a,c is a clear evidence that the wall is strong in resisting bending and weak in shear and shear failure mode would occur prior to flexural failure mode
4. **Therefore, the DPO panel's statement that the wall [ ] a,c is an evidence that all the three analyses did not include the [ ] a,c in their concrete material model in their AIA analyses, and thus the wall did not fail in punching shear and such an analysis result is <u>false</u> and <u>unsafe</u>**

Falsehood of DPO Panel's Evaluation on AIA (continued)

5. The DPO panel report, dated February 24, 2014, stated "The panel notes that RES did not perform a static or dynamic LS-DYNA benchmark of steel plate and concrete composite panels like the SC module …the Panel has confidence in RES's ability to adequately model the SC wall of the SB and perform aircraft impact analyses"
6. Benchmarking the applicability and adequacy of a computer code is an necessary step and the most important function prior to its use and is required by the AIA rule. **The DPO panel placed its blind trust in RES knowing benchmarking was not done is incorrect and dangerous**
7. The DPO panel had the shear-deformation relationship (slide #26) for the SC wall element and chose not to make sure that all the three analyses did include this shear-deformation relationship material model in their computer codes is a major mistake
8. **Had this shear-deformation relationship been included in the three AIA analyses, punching shear failure mode (perforation) would have definitely occurred prior to the [ ] a,c**

Falsehood of DPO Panel's Evaluation on AIA (continued)

9. The DPO panel attributed the curvature of the AP1000 shield building wall being the major factor in resisting the aircraft impact load
10. However, research in concrete (both in theory and test) indicated that loading on the positive side of curvature, such as aircraft impact on the shield building, would have less shear strength than that of a flat surface[1]
11. ACI Code's formula reveal that thickness of concrete wall, slab, or footing is the most important factor in punching shear strength because it is proportional to the square power of its thickness (slide #84) - a 6 feet thick EPR wall would have 4 times more punching shear strength than that of the 3 feet thick of AP1000 shield building wall that is only half as thick as EPR
12. The new shield buildings (German Convoi NPPs, EPR, ATMEA1, EC6) that were identified by the Canadian nuclear regulators have curvatures, similar to that of AP1000 shield building, but all required 6 feet thick walls

Footnote:

1. "Shear Strength of Arch-Shaped Members without Transverse Reinforcement," by Stefano Campana, Miguel fenandez, and Aurelio Muttoni, ACI Structural Journal/May-June 2014

Falsehood of DPO Panel's Evaluation on AIA (continued)

13. With a blind trust in RES analysis with no benchmarking, and no knowledge of benchmarking on the Westinghouse and its consultant's codes, the DPO panel solely relied on these three analyses results and ignored the ACI Code's requirement on punching shear failure, and other analyses resulted in 6 feet thick shield building walls, including EPR AIA that the NRC had reviewed and approved. The DPO panel's approach is **illogical and unsafe**.

14. My DPO also stated that the failure of the anchorage of the PSC tank could cause the heavy tank tumbling down and crash the steel containment and damage the core and result in an early and large uncontrolled release of radiation into the environment

Falsehood of DPO Panel's Evaluation on AIA (continued)

15. The DPO panel report, dated October 30, 2014, stated

(1) Westinghouse stated: "NRC Staff did not review the PCS tank aircraft impact assessment." and "NRC Staff had not reviewed the steel containment assessment for the potential falling debris"

(2)The NRO staff stated that it performed an audit of the design of the SB PCS tank in June 2011 (ADAMS Accession No. ML111810034),

(3) However, the report does not address the inspection of the PCS tank or the steel containment structural assessment for the aircraft impact and shock loads.

(4) The Panel interviewed and interacted with the NRO/DE/ SEB Branch Chief, but was unable to get satisfactory answers

(5) In its conclusion, the Panel finds no reason to doubt the assessments made by the staff and WEC on this issue.

16. When a new AIA inspection team arrived at Westinghouse on July 27, 2015, it had nothing to inspect because there was nothing

Falsehood of DPO Panel's Evaluation on AIA (continued)

17. With the information (1) provided by the Westinghouse that the NRC staff did not review the PCS tank and the steel containment, (2) the DPO panel's own review results that contradicted with the NRO staff's claims that it had reviewed and documented, and (3) no satisfactory answers from the NRO/DE/ SEB Branch Chief on the AIA inspection, the logical conclusion is that the NRO staff might not have performed the inspection as it claimed it had. No fair-minded people would have concluded like the DPO panel did: **"The Panel finds no reason to doubt the assessments made by the staff on this issue."**
18. In addition to the issue that the NRO/DE/SEB staff did not properly conduct an AIA inspection, it appeared that Westinghouse did not perform a proper AIA either and that was probably why the new NRC team had nothing to inspect on July 27, 2015
19. The DPO panel's conclusion is **false and unsafe**

Common Sense

1. Under the same NRC rules, the same seismic intensity of 0.3g, and the same ACI Code, the shield building wall thickness should be about the same for both AP1000 and EPR
2. The addition of the PCS tank and of the irregular shape of anchorage (slide #15) should generate additional shear stresses in AP1000 shield building wall during earthquakes but not in EPR wall
3. The possibility or risk of containment breach and core damage resulting from the collapse of the PCS tank due to its anchorage failure, or wall failure, caused by earthquakes or aircraft impact, is real for AP1000, but not for EPR
4. The possibility or risk of containment breach and core damage as a result of aircraft impact is real for the 3 feet thick concrete wall and a steel containment inside it, but not for the EPR due to its 6 feet thick concrete shield building wall and a concrete containment inside it (had an aircraft breached the 6 feet thick EPR shield building wall, the residual force or energy would not be able to damage the concrete containment, but had an aircraft breached the 3 feet thick shield building wall the residual force and energy may cause damage to the AP1000 steel containment)

Based on the above information, common sense would expect and conclude that a thicker concrete wall for the AP1000 shield building would be required. However, the wall thickness design went to the opposite direction, which is a clear evidence that the design is mistaken and unsafe

Mistakes of Civil/Structural Design and their Consequence

Information from “A question of Ethics: The lesson of Katrina,” by Tara Hoke, ASCE’s general counsel
ASCE Civil Engineering Magazine, July/August 2015

Major design mistakes in Katrina disaster

1. Overestimated the average soil strength by 30 percent
2. Underestimated wind speed by 20% (actual wind speed = 125 mph, and the design wind speed = 100 mph)

Consequence

More than 1,800 fatalities and more than $100 billion in damage

ASCE’s External Peer Review Put it This Way:

In the face of pressure to save money or to make up time, engineers must remain strong and hold true to the requirements of the profession’s canon of ethics, never compromising the safety of the public.

NRO/DPO Panel Took Similar Actions that the Corps Did and More

1. An overestimated 30% soil strength, and underestimated 20% of design wind speed were the main cause of the Katrina disaster. Similarly the maximum RLE shear stress is [ ] [a,c] ACI Code's shear strength and the maximum SSE shear stress is also [ ] [a,c] the NEHRP's design shear strength in the AP1000 shield building wall, but the NRO and the DPO panel see no danger and would not address them
2. More dangerously, the majority of wall elements in the AP1000 shield building wall are brittle (slide #26), which is unprecedented in modern concrete structural design, prohibited by the ACI Code (slide #21), and unacceptable by the ASCE training course (slide #22). However, the NRO (slides #35 and #39) and the DPO panel considered it acceptable
3. When the National Weather Service raised its projected maximum wind speed to 151 to 160 mph, the Corps failed to update its original design wind speed of 100 mph. While all new shield building design in the world resulted in a six (6) feet thick concrete wall, including EPR that the NRC reviewed, based on new design goal and rules, the NRO and the DPO panel see no abnormality and danger in the AP1000 shield building wall design that resulted in three (3) feet thick concrete wall
4. **While the UK nuclear regulators questioned how the AP1000 shield building could sustain the NRC's RLE event, and the Canadian nuclear regulator's statement that they did not believe the three (3) feet thick concrete wall of the AP1000 shield building is adequate, the NRO and the DPO panel just looked the other way and kept quiet**

Appeal to EDO

- Be Faithful to Our Mission:

  To Protect the Pubic Health, Safety, Welfare, and the Environment

- Have Courage to Enforce Rules for New Reactors:

  1. Seismic Margin Analysis
  2. Aircraft Impact Analysis, and

  the design/licensing basis of the AP1000 shield building - ACI Code, for the Design of AP1000 shield building

- Have courage to admit past mistakes and right the wrong

The End

And

Thank You

Document 10: DPO Appeal Decision

February 9, 2017

MEMORANDUM TO: John S. Ma
Senior Structural Engineer
Office of New Reactors

FROM: Victor M. McCree ***/RA/***
Executive Director for Operations

SUBJECT: DECISION ON DIFFERING PROFESSIONAL OPINION APPEAL CONCERNING THE AP 1000 SHIELD BUILDING (DPO-2012-002)

The purpose of this memorandum is to discuss my decision on your appeal submitted on March 31, 2015, on the subject differing professional opinion (DPO). The DPO program is addressed in Management Directive 10.159, “The NRC Differing Professional Opinions Program.” Your DPO dated July 5, 2012, involved the U.S. Nuclear Regulatory Commission (NRC) certification of the AP 1000 design. Specifically, you raised questions concerning the AP 1000 Shield Building design. You were concerned that the AP 1000 shield building design was unsafe based on the following assertions: (1) the certified AP 1000 shield building does not meet the NRC’s seismic margin requirement, (2) there is no adequate demonstration that the shield building meets the General Design Criterion (GDC) 2 requirement, (3) the NRC’s conclusion that the aircraft missile would not penetrate the AP 1000 shield building wall is illogical, and (4) the shield building wall is insufficiently strong and ductile to resist earthquakes or aircraft impact.

On August 29, 2012, the Director of the Office of New Reactors (NRO) tasked an ad-hoc review panel to review your DPO. On September 20, 2012, the Director of NRO issued a memorandum changing the membership of the ad-hoc review panel. The ad-hoc review panel issued a detailed report on February 24, 2014. In this report, the panel concluded that:

- the AP 1000 SB [shield building] does meet the NRC’s seismic margin requirement;
- the AP 1000 Shield Building wall, constructed of steel-concrete (SC) modules, would not be perforated by the aircraft impact at room temperature. This conclusion was based on three independent AIAs [aircraft impact assessments].
- the AP 1000 SB possesses both sufficient strength and ductility to be appropriately resistant to seismic excitation and aircraft impact per NRC requirements; and
- on the basis of conclusions in the first and third items above, the Panel concluded that the AP 1000 SB meets the requirements of General Design Criterion 2, “Design bases for protection against natural phenomena.”

The DPO Panel report also raised a number of unresolved questions that were referred to the staff for follow-up. Some examples included:

- Discrepancies in computations of High Confidence Low Probability of Failure (HCLPF) values for the shield building.

- Panel members were of different opinions concerning the effects of temperature on the AIA of the shield building with member [redacted] agreeing with the submitter that "the ability of the shield building SC wall to resist aircraft impact under cold weather has not been substantiated."
- Some items that had not been inspected by the staff or for which insufficient information was available (i.e. PCS tank AIA and benchmarking of computer models for AIA)
- Observations concerning the shear reinforcing details at the connection of the PCS tank to the conical roof
- Observations concerning design and constructability of the tapered SC wall in the air-inlet region
- Follow-up on the design and analysis of the PCS tank for design basis loads.

Based on the observations of the DPO panel in their report, the NRO Director convened meetings to hold detailed discussions of the technical issues on May 19 and June 5, 2014. Subsequently, the NRO Director required staff to develop a report summarizing the deliberations for the Panel's review. This report was completed on July 24, 2014 and was forwarded to the Panel with a tasking to supplement their original report. The Panel's supplemental report, dated October 30, 2014, concluded that the staff had followed-up on the Panel's recommendations and found that overall the staff's resolution was acceptable with three issues that could merit further consideration.

Based on the DPO Panel's Report and Supplemental Report, the Director of NRO issued a Director's Decision on February 27, 2015, in which he agreed with the Panel's conclusions that the AP 1000 shield building design meets the NRC's seismic and aircraft impact assessment requirements, and so provides reasonable assurance of public health and safety. He also directed the NRO staff to follow-up on the three issues identified in the Panel's supplemental report. Since that time, these three issues have been satisfactorily closed out.

On March 31, 2015, you submitted an appeal to my predecessor, Mark Satorius, on the NRO Director's decision. Your appeal addressed three major areas of concern: 1) Shear ductility [out-of-plane] of the shield building SC wall; 2) Shear strength [in-plane] of the shield building SC wall; and Punching shear failure of shield building SC wall due to an aircraft impact.

Thank you for your professionalism and your willingness to challenge and raise these concerns with your colleagues and management. A staff that exhibits a questioning attitude and examination of NRC technical positions and licensing approaches is essential in maintaining a healthy NRC regulatory program. I also appreciated the significant amount of time, documentation, and analyses you offered to support your differing professional opinion on the AP 1000 shield building. I recognize that the time required to review and render a decision on your DPO appeal has been lengthy. However, it was important for me to fully understand your concerns, examine the record, independently verify aspects discussed in your DPO, and follow up on items identified by the NRO Director to address some of your concerns.

Attached is a detailed description of my evaluation, findings, and decision on the three key appeal issues. To summarize, I support the decision by the DPO Panel and the Director of NRO that the AP 1000 Shield Building design meets the U.S. Nuclear Regulatory Commission (NRC) requirements, and so provides reasonable assurance of public health and safety. While I agree with the decision regarding the AP 1000 Shield Building, as described in the attached detailed decision analysis, I have also identified areas for follow-up by NRO staff. Specifically, I am directing the NRO staff to:

a. Verify that the out-of-plane shear strength of both the SC and RC portions of the shield building wall in the areas where there is a potential for plastic hinge is larger than the shear force associated with the flexural over-strength of the SC/RC wall.
b. Verify the validity of the inertia force distribution used in the AP 1000 shield building nonlinear static pushover analysis to provide assurance that the shield building dynamic behavior has been adequately captured.
c. Verify the reasonableness of the results of the AP 1000 shield building nonlinear static pushover analysis in comparison with the nonlinear response history analysis method.

Based on the consideration of all the information provided in my appeal review, I do not believe this follow-up activity is a safety issue that is significant enough to require immediate intervention in the current AP 1000 construction activities.

In accordance with MD 10.159, a summary of this DPO appeal decision will be included in the Weekly Information Report posted on the NRC's public Web site to advise interested employees and members of the public of the outcome. If you request, the DPO case file will also be publicly available, subject to a review consistent with all agency requirements. The DPO Program Manager will contact you separately on this issue.

Thank you again for your attention and dedication to this important matter. Please do not hesitate to contact me if you have additional concerns.

Attachment:
Executive Director for Operations Decision
on Appeal to DPO-2012-002

cc:
V. Ordaz, NRO
P. Holahan, OE
M. Weber, RES
R. Pedersen, OE
M. Sewell, OE
J. Monninger, NRO
H. Bell, OIG

MEMORANDUM: DECISION ON DIFFERING PROFESSIONAL OPINION APPEAL CONCERNING THE AP 1000 SHIELD BUILDING (DPO-2012-002) DATED FEBRUARY 9, 2017.

ADAMS Accession No: ML17033B675 (Pkg); ML17033B676 (Memo); ML17033B680 (Attachment)

OFFICE	NRR/DE	NRR/DE	OEDO/AO	EDO
NAME	FFarzam (via email)	GThomas (via email)	JJolicoeur	VMcCree
DATE	1/31/17	1/31/17	2/ /17	2/09/17

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Executive Director for Operations—Decision on Appeal to DPO-2012-002

1.0 Background

The differing professional opinion (DPO) submitted on July 5, 2012, the DPO Panel Report issued on February 24, 2014 and the DPO Panel Supplemental Report issued on October 30, 2014, provide a significant discussion on the background and history surrounding this DPO. A summary follows below.

The AP 1000 shield building is a safety-related seismic Category I structure that provides shielding for the containment vessel (CV) and the radioactive systems located in the CV; protects the containment from external events; provides the required shielding for radioactive airborne materials that may be dispersed in the containment; supports the passive containment cooling water storage tank; and provides for natural air circulation cooling for the CV.

The AP1000 shield building design as described in the AP1000 Design Control Document (DCD), Revision 15, was a reinforced concrete (RC) design. In AP1000 DCD Revisions 16 and 17, a new shield building design was proposed. The revised design incorporates a steel concrete composite (SC) structure as opposed to the traditional RC shield building construction.

The key features of the revised shield building are: a cylindrical wall which comprises the bulk of the structure constructed of SC modules; a conical RC roof structure with an integral RC water tank which contains approximately 7 million pounds of water; a tension ring at the intersection of the roof with the cylindrical wall consisting of a built-up closed section of steel plates filled with concrete; and mechanical connections that join the SC wall to the basemat and to the RC cylindrical portion of the shield building.

The tension ring is designed as a steel structure in accordance with the American National Standards Institute/American Institute of Steel Construction (ANSI/AISC) Standard N690. The steel frame for the roof is designed to ANSI/AISC N690. The concrete roof is designed to American Concrete Institute (ACI) 349 requirements without credit for the steel plate on the bottom of the concrete. The SC modules have not been used previously in nuclear construction in the United States.

In the initial design proposed for the new shield building, the SC wall module for the 3-foot thick cylindrical wall consisted of steel faceplates with attached [ ] [a,c] long steel studs which were embedded in the 35-inch thick concrete fill between the two faceplates. In a letter dated October 15, 2009, the NRC staff determined that this design would require modifications to ensure its ability to perform its safety function under design basis loading conditions. During the review, NRC staff concerns focused particularly on the lack of transverse reinforcement that would tie one faceplate to the opposite faceplate to ensure that the SC modules would function as a unit for either out-of-plane demands or in-plane demands. The DPO Submitter played a key role in articulating the staff's concerns in this area. Westinghouse Electric Company (WEC) developed a revised design for the shield building that added tie bars welded to opposite faceplates in the SC wall modules, and also revised the design of the ring girder and the connections between the SC wall module and the RC wall. The revised SC wall module has thicker faceplates, as well as tie bars between the faceplates to help ensure that the module acts as a composite unit with increased out-of-plane shear strength.

WEC used ACI 349, a design code for nuclear safety-related RC structures, to guide their design of the SC cylindrical wall modules since a US industry code or standard for SC construction did not exist at the time. Even though the scope of ACI 349 does not include SC construction, the underlying design philosophy, elastic behavior and strength for design basis loads and resilience through ductility for beyond design-basis loads, does apply. Also, the underlying assumptions on composite behavior of steel and concrete materials in RC structural elements do apply to SC structural elements.

WEC conducted a testing program at Purdue University. The tests were intended to demonstrate that ACI 349 could be used to predict the out-of-plane shear strength, flexural capacity, and in-plane shear strength of SC structures and to investigate the failure behavior of the SC modules. The test results were also used to benchmark the finite element analyses performed to support the design of the AP1000 shield building.

NRO requested that the Office of Nuclear Regulatory Research (RES) provide assistance in evaluating the structural analysis, design, construction, and inspection methods for the AP1000 shield building. The findings in the RES report (ADAMS Accession No. ML103080129) were used to inform the evaluation of the shield building design by the staff of NRO. RES staff assessed and consolidated the inputs from outside experts and performed their own independent assessment to develop their report.

2.0 Summary of DPO

DPO-2012-002 was filed on July 5, 2012 raising the following safety issues about the design of the AP1000 Shield Building (SB):

1. the certified AP1000 SB does not meet the NRC’s seismic margin requirement;
2. there is no adequate demonstration that the SB meets the General Design Criteria 2 (GDC 2) requirement;
3. the NRC’s conclusion that the aircraft missile would not penetrate the AP1000 SB wall is illogical; and
4. the SB wall is insufficiently strong and ductile to resist earthquakes or aircraft missiles.

The DPO Panel established by the Director, Office of New Reactors studied the issues raised in the DPO. In their report, the panel grouped the identified safety concerns into two broad categories (beyond design-basis demand and capacities, and design-basis demand and capacities) and reordered them as follows:

1. Safety concern #1 – The certified design of the AP1000 SB does not meet the NRC’s seismic margin requirements;
2. Safety concern #2 – The NRC staff’s conclusion that aircraft missile would not penetrate the AP1000 SB wall is not logical;
3. Safety Concern #3 – The certified SB wall does not possess sufficient strength and ductility to resist an earthquake and/or aircraft missile impact loading that is specified by the NRC; and

4. Safety concern #4 – The Certified SB design does not meet GDC 2 requirements.

3.0 Establishment of the DPO Panel

On August 29, 2012, the NRO Director established an ad-hoc panel to review the issues raised in the subject DPO. On September 20, 2012, the Director of NRO issued a memorandum changing the membership of the panel. In his September 20, 2012 memorandum, the Director of NRO charged the panel to:

- Review the DPO submittal to determine if sufficient information has been provided to undertake a detailed review of the technical issues.
- Meet with the submitter, as soon as practicable, to ensure that the DPO Panel understands the submitter's concerns and scope of the issues.
- Promptly after the meeting, document the DPO Panel's understanding of the submitter's concerns, provide the Statement of Concerns (SOC) to the submitter, and request that the submitter review and provide comments, if necessary.
- Maintain the scope of the review within those issues defined in the original written DPO and confirmed in the SOC.
- Consult with me [Director of NRO] as necessary to discuss schedule-related issues, the need for technical support (if necessary), or the need for administrative support for the DPO Panel's activities.
- Perform a detailed review of the technical issues and conduct any record review interviews, and discussions deemed necessary for a complete, objective, independent, and impartial review. In particular, since this concern relates to a Commission-approved regulation (i.e. the AP1000 Design Certification, Title 10 of the *Code of Federal Regulations* (10 CFR), Part 52 Appendix D), the panel should evaluate the technical merits of the DPO to determine if there is a sufficient basis to initiate the rulemaking process to modify 10 CFR 52 Appendix D. The panel should use the criteria in 10 CFR 52.63 "Finality of standard design certifications" in considering the need for a change to the AP1000 Design Certification.
- Issue a DPO Panel report, including conclusions and recommendations to me [Director of NRO] regarding the disposition of the issues presented in the DPO. The report should be a collaborative product and include all DPO Panel member's concurrence. Follow the specific processing instructions for DPO documents.

The panel found the technical issues to be sufficiently complex that they requested and NRO authorized the use of two contract technical experts to support the deliberations. Throughout the panel's review, the DPO submitter had significant interactions with the panel as described in the panel's report.

4.0 DPO Panel Report

The DPO Panel performed a review of the technical issues as described in the SOC and conducted document reviews, interviews and discussions necessary for their review. The Panel also reviewed all email communications of technical information that the DPO submitter provided to the Panel for its consideration. Their report, dated February 24, 2014

(ML14057A580) provided the Panel's conclusions and recommendations and the technical bases supporting the conclusions and recommendations. In this report, the panel concluded that:

- Safety Concern #1 - the AP1000 SB does meet the NRC's seismic margin requirement;
- Safety Concern #2 – the AP1000 Shield Building shell, constructed of SC modules, would not be perforated by the aircraft impact at room temperature. This conclusion is based on three independent AIAs [aircraft impact assessments].
- Safety Concern #3 – The AP1000 SB possesses both sufficient strength and ductility to be appropriately resistant to seismic excitation and aircraft impact per NRC requirements; and
- Safety Concern #4 – On the basis of conclusions regarding Safety Concerns #1 and #3 above, the AP1000 SB meets the requirements of General Design Criterion 2, "Design bases for protection against natural phenomena."

The DPO Panel report also raised a number of unresolved questions that were referred to the staff for follow-up. Some examples included:

- Discrepancies in computations of High Confidence Low Probability of Failure (HCLPF) values for SB.
- Panel members were of different opinions concerning the impact of temperature effects in the AIA evaluation of the SB walls with one member of the panel agreeing with the submitter that "the ability of the shield [building SC] wall to resist aircraft impact under cold weather has not been substantiated."
- Some items that had not been inspected by the staff or for which insufficient information was available (i.e. PCS [Passive Cooling System] tank AIA and benchmarking of computer models for AIA)
- Observations concerning the shear reinforcing details at the connection of the PCS tank to the conical roof
- Observations concerning design and constructability of the tapered SC wall in the air-inlet region
- Follow-up on the design and analysis of the PCS tank for design basis loads.

5.0 DPO Panel Supplemental Report

Based on the observations of the DPO panel in their report, the NRO Director convened meetings to hold detailed discussions of the technical issues on May 19 and June 5, 2014. Subsequently, the NRO Director required staff to develop a report summarizing the deliberations for the Panel's review. This report was completed on July 24, 2014 and was forwarded to the Panel with a tasking to supplement their original report. The Panel's supplemental report, dated October 30, 2014, (ML14307A870) concluded that the staff had followed-up on the Panel's recommendations and found that overall the staff's resolution was acceptable with three issues that could merit further consideration.

6.0 NRO Director's Decision on DPO

Based on the DPO Panel's Report and Supplemental Report, the Director of NRO issued a Director's Decision on February 27, 2015, in which he agreed with the panel's conclusions that the existing AP1000 shield building design meets the NRC's seismic and aircraft impact assessment requirements, and so provides reasonable assurance of public health and safety without changing that design. He also directed the NRO staff to follow-up on the three issues identified in the Panel's supplemental report, specifically:

- Confirm and document that WEC QA process has adequately addressed discrepancies in seismic margin estimates;
- Perform an inspection of WEC's aircraft impact assessment documentation, focusing on the PCS tank and any postulated debris impact on containment; and
- Clarify documentation of NRC staff review of the AP1000 shield building for SRP sections 3.7.2.II.11 and 3.7.2.II.12, including augmenting its documentation in a memorandum.

The NRO staff has completed the follow-up actions requested in each of the bullets above (ADAMS Accession No. ML 15134A037, ML16167A254, ML16099A049, ML16146A662, and ML15180A061).

7.0 DPO Appeal

On March 31, 2015, the DPO submitter filed an appeal on the Director's Decision. The DPO appeal addressed three major areas of concern:

(1) Shear ductility [out-of-plane] of the shield building wall – the DPO panel's method to determine shear ductility is incorrect and dangerous to safety;
(2) Shear strength [in-plane] of the shield building wall;
(3) Punching shear failure of the shield building SC wall due to an aircraft impact.

I will address these concerns further in my review and decision below (see Section 9).

8.0 Statement of Views by the Director of NRO on Appeal

On July 31, 2015, the Director of NRO provided his statement of views (SOV) on the DPO appeal to me. In his SOV he stated that, based on his review of the appeal and consultation with the Ad-hoc DPO Panel and staff experts, he had "concluded that the appeal does not provide any new information or new interpretation of existing information that changes the positions described in my Director's Decision." The NRO Director's SOV referenced a detailed analysis (ML15195A41) of the issues raised in the DPO Appeal and the resolution of these issues as addressed in the original DPO decision and the supporting DPO Panel reports.

9.0 EDO Review and Decision

When OEDO received the DPO appeal, my predecessor, Mark Satorius, with my concurrence, initiated a review of relevant information related to DPO 2012-002. To that end, he assigned a senior member of my staff to form a working group including two senior structural engineers with technical qualifications in the areas of concrete codes as well as aircraft impact analysis who were not involved in the AP1000 shield building review and the review of the original DPO. This working group has provided technical assistance to this review. I reviewed a number of documents, including but not limited to, the original DPO, the DPO Panel's report, the DPO Panel's supplemental report, the NRO Office Director's decision, the DPO appeal of the decision, and the NRO Office Director's SOV on the contended issues in the DPO appeal. To understand the issues fully, I met with the DPO panel on December 19, 2016. The DPO appeal working group and I met with you on June 21, 2016, and listened carefully to your points. I also considered additional information you provided to me before, during, and after the meeting. I had multiple discussions with the former Director of NRO to address your concerns.

I want to clarify that my decision considers the technical merits of your DPO, as well as our regulatory framework for ensuring safety. Below is my review and decision on the concerns raised in the appeal:

Appeal Concern 1: Out-of-Plane Shear Strength and Ductility of Shield Building SC Wall

The DPO appeal includes the following main technical issues regarding the out-of-plane shear strength and ductility of the AP1000 shield building SC wall:

I. The AP1000 shield building SC wall element is brittle [non-ductile] in shear, and thus does not meet the requirements of ACI Code, which is the design basis of the AP1000 shield building.
II. The DPO panel considered the shield building SC wall to be ductile with a ductility ratio of [ ] a,c in shear meeting the required ductility ratio of [ ] a,c that was jointly established by the NRC and Westinghouse.

You also included Dr. Vecchio's statements in his July 9, 2010 letter, Dr. Hsu's statement in his November 14, 2010 letter, and Dr. Varma's statement regarding non-ductile behavior of test specimens with tie-bars spaced at [ ] a,c in support of your above position.

Enclosure 1 to this memorandum provides the DPO Appeal working group perspectives which evaluated the technical issues raised in the DPO appeal regarding the out-of-plane shear strength and ductility of the shield building SC wall.

Based on my review of the information, findings and insights in Enclosure 1 to this memorandum, NRO Office Director's (OD) decision, OD's statement of views on DPO Appeal, DPO Panel report and DPO Panel supplemental report, I concluded the following:

I. The test specimens with [ ] a,c tie spacing exhibited non-ductile behavior for low shear span-to-depth ratios and this has been acknowledged in the NRO staff SER and the DPO Panel report. The DPO panel report did not consider the out-of-plane test with shear span/depth ratio of [ ] a,c as a shear test with a ductility ratio of [ ] a,c The DPO panel's observation was mainly trying to indicate that the effective shear-span-to-depth ratio away from the connection areas (discontinuities) is high; therefore, the shear failure as observed in the test with low span-to-depth ratio is not applicable to areas with [relatively wide] tie rod spacing.
II. The methodology used for the AP1000 shield building SC wall design is similar to "capacity design" methodology. The applicant provided smaller tie bar spacing in those areas of the SC wall where there is a potential for plastic hinge, for beyond design basis

seismic load conditions, to ensure ductile failure and demonstrated that in the areas with [ ] a,c spacing the out-of-plane shear demand to-capacity is low with ample margin. The use of design methodology similar to “capacity design” is reasonable and consistent with the industry standards and published technical literature.

III. The response of the DPO Panel and the NRO staff to your assertion regarding the out-of-plane shear capacity and ductility of the SC wall was reasonable. However, to confirm the thoroughness of documentation supporting my decision, I am directing the NRO staff to complete the following actions:

   a. Verify that the out-of-plane shear strength of both the SC and RC portions of the shield building wall in the areas where there is a potential for plastic hinge is larger than the shear force associated with the flexural over-strength of the SC/RC wall.

   b. Verify the validity of the inertia force distribution used in the AP1000 shield building nonlinear static pushover analysis to provide assurance that the shield building dynamic behavior has been adequately captured.

   c. Verify the reasonableness of the results of the AP1000 shield building nonlinear static pushover analysis in comparison with the nonlinear response history analysis method.

Appeal Concern 2: In-Plane Shear Strength of the Shield Building SC Wall

The DPO appeal includes the following main technical issues regarding the in-plane shear strength design of the AP1000 shield building SC wall:

a) The DPO panel’s evaluation on the shear strength of the AP1000 shield building relied solely on Mr. Loring Wyllie’s [DPO Panel consultant] opinion and judgment and considered the use of [ ] a,c conservative, instead of the ACI Code’s limit of 8√ fc’, which is the design basis of the AP1000 shield building.

b) The shear stress due to review level earthquake (RLE) exceeds the value of [ ] a,c contradicting the DPO Panel Report statement that the AP1000 shield building wall has sufficient shear strength to resist RLE satisfying NRC seismic margin requirements.

c) There are modeling deficiencies (accidental torsion, P-delta effect, and irregular zigzag connection between the SC and RC wall) of the AP1000 shield building that could underestimate the magnitude of shear stress in the wall.

The DPO Appeal working group members prepared Enclosure 2 to this memorandum addressing the main technical issues in the DPO appeal relative to the in-plane shear capacity of the SC wall.

Based on my review of the information, findings and insights in Enclosure 2 to this memorandum, the NRO Office Director’s (OD) decision, the OD’s statement of views on the DPO Appeal, the DPO Panel report and the DPO Panel supplemental report, I agree with the OD’s and DPO Panel’s conclusion that the AP1000 shield building SC wall has sufficient in-plane shear capacity.

Appeal Concern 3: Punching Shear Failure of the Shield Building SC Wall Due to an Aircraft Impact

The DPO appeal stated that the design of the AP1000 shield building SC wall is strong, with a great amount of energy absorption capability, in resisting bending (flexure) but weak and small energy absorption capability in shear, [ ] a,c (indicated by the "red curve" in a figure in DPO-2012-002). The appeal asserted that the actual local failure mode of the wall, as a result of aircraft impact, should be punching shear instead of [ ] a,c as was evaluated and which controlled the shield building wall thickness for the design-basis hurricane-generated automobile missile. As such, the DPO appeal concluded that the shield building SC wall must be much thicker to adequately resist local aircraft impact loading in a punching-shear failure mode.

The DPO Appeal also included the following arguments in support of your above view:

a) Citing information on slide 69 of your presentation to the NRO Office Director and DPO Panel on September 8, 2014 (ML15078A045), with the exception of the AP1000 shield building, the required concrete wall thickness for not being punched through by aircraft in the other designs or analyses listed in the slide was about 6 feet. Further, the punching shear strength for a concrete slab, footing or wall is related to the square power of thickness and, therefore, the [ ] a,c thick AP1000 SC wall has only [ ] a,c of the punching shear strength of a 6-ft thick wall (e.g. EPR shield building), which is a reason to doubt the adequacy of the AP1000 SC wall thickness.
b) The AP1000 shield building wall is neither simply supported nor fixed end supported in the circumferential direction as considered in Dr. Rashid's [DPO Panel consultant] arch-shaped mathematical model to demonstrate the role of curvature (versus flat walls) in providing superior impact resistance of the shield building wall, with its fixity being some value in between. Therefore, the DPO Panel's explanation that the arch-action from curvature and support conditions of the shield building were responsible for its great resistance to perforation by punching shear under aircraft impact is not convincing.
c) The DPO Panel consultant's aircraft impact analyses results from the computer codes indicated a [ ] a,c failure mode, suggesting that the shear failure mode (the red curve) had not been, or had been incorrectly, coded into the computer analytical models.

The DPO Appeal working group prepared Enclosure 3 to this memorandum addressing the main technical issues in the DPO appeal relative to punching-shear failure of the SC wall under beyond design-basis aircraft impact.

Based on my review of the information, findings and insights in Enclosure 3 to this memorandum, the NRO Office Director's (OD) decision, the OD's statement of views on DPO Appeal, the DPO Panel report and the DPO Panel supplemental report, I agree with the OD's and DPO Panel's conclusion that the AP1000 shield building SC wall will not be perforated by the NRC-specified beyond design-basis aircraft impact in a punching-shear failure mechanism.

10.0 Conclusion

Thank you for your professionalism in raising this differing professional opinion on an important safety topic. Given the uniqueness of this design and the fact that this is the first application of steel concrete composite design that has been reviewed by NRC staff, it is important that the staff assure itself that this design meets the NRC's regulations and expectations for safety. This issue was complicated by the lack of US approved consensus codes or standards for steel concrete composite construction in nuclear safety related applications.

After careful review, I support the decision by the DPO Panel and the Director of NRO that the AP1000 Shield Building design meets the NRC requirements and provides reasonable assurance of public health and safety. While I agree with the decision regarding the AP1000 Shield Building, as noted in section 9.0 above, I have also identified several items for follow-up by the NRO staff. Specifically, I am directing NRO to: (1) verify that the out-of-plane shear strength of both the SC and RC portions of the shield building wall in the areas where there is a potential for plastic hinge is larger than the shear force associated with the flexural over-strength of the SC/RC wall; (2) verify the validity of the inertia force distribution used in the AP1000 shield building nonlinear static pushover analysis to provide assurance that the shield building dynamic behavior has been adequately captured; and (3) verify the reasonableness of the results of the AP1000 shield building nonlinear static pushover analysis in comparison with the response history analysis method.

In accordance with MD 10.159, a summary of this DPO appeal decision will be included in the Weekly Information Report posted on the NRC's public web site to advise interested employees and members of the public of the outcome.

This enclosure provides the DPO Appeal working group perspectives on the out-of-plane shear strength and ductility of the shield building SC wall based on a review of information included in the submitter's DPO appeal (Reference 1), the DPO Panel report (Reference 2), the DPO Panel supplementary report (Reference 3), and the NRO staff SER (Reference 4). This enclosure does not address the Aircraft Impact Assessment (AIA). The AIA is included in Enclosure 3.

As directed in the EDO memorandum (Reference 5), in light of the significant work already conducted in this case, the limited scope and primary focus of this review was to apply the reasonableness test, as opposed to initiating an independent re-review, to issues raised in the DPO appeal against the responses provided in the DPO Panel reports and the NRO Office Director's findings to determine whether the issues were addressed sufficiently. Therefore, the perspectives of the DPO Appeal working group should not be construed as a peer review of the AP1000 shield building design.

Considering the above, the following excerpts from References 1 through 4 provide background of the main technical issues regarding the out-of-plane shear strength and ductility of the shield building SC wall.

a) Submitter's Assertion (DPO Appeal, Reference 1)

   I. The AP1000 shield building SC wall element is brittle in shear, and thus not meeting the requirements of ACI Code, which is the design basis of the AP1000 shield building.

   II. The DPO panel considered the shield building SC wall to be ductile with a ductility ratio of [ ] [a,c] in shear.

   III. Dr. Vecchio stated in his July 9, 2010 letter:

   "The test beams representing the Shield Building wall detail with cross-ties spaced at [ ] [a,c] tested in out-of-plane shear (OOPS) at Purdue, exhibited shear-critical brittle failures. Westinghouse Electric Corporation (WEC) contends that, even so, the result is acceptable because code-calculated strengths were achieved by the test specimens and that the design shear forces fall well short of these strengths. However, the load contour plots provided by WEC show that, in general, the moment demand-to-capacity ratios are significantly lower than the shear demand-to-capacity values. In other words, for out-of-plane action, shear demand is more critical than flexural demand; if the loads were increased in fixed proportion, the wall would sustain a brittle shear failure first. This violates the intent of Clause 21.3.4.1."

   ''Although ACI 349 does not define specific target levels for ductility, it is worth noting that it implicitly requires that the failure mechanism be ductile regardless of the magnitudes of the actual design loads. Thus, for shear design of flexural members, Clause 21.3.4.1 states that "…the design shear force shall be determined from consideration of the statical forces on the portion of the member between the faces of the joints. It shall be assumed that moments of opposite sign corresponding to the probable flexural moment strength act at the joint faces…" "In other words, the design shear force is dictated by the flexural capacity of the member, ensuring that a ductile flexural failure occur[s] before a brittle shear failure can develop. In the case of the SB wall, the [ ] a,c steel faceplates create a large moment capacity, and thus the shear capacity required to maintain a ductile failure mechanism is high, regardless of the actual out-of-plane shear forces acting."

IV. Dr. Hsu, stated in his Nov. 14, 2010 letter:

"The tie bars proposed by WEC are [ ] a,c in spacing in both the directions of height and circumference. However, tests at Purdue University [ ] a,c

V. Dr. Varma, who conducted the tests for Westinghouse, stated that he was sure that all the test specimens, with tie-bar spaced at [ ] a,c

b) DPO Panel Report (Reference 2) and DPO Panel Supplementary Report (Reference 3)

I. Excerpt from Section 4.2.3.1 of DPO Panel Report

"For a shear span to depth ratio of [ ] a,c a very ductile flexural failure occurred [ ] a,c and for a shear span to depth ratio of [ ] a,c a non-ductile shear failure occurred. The red curve shown in the figure is for a SC panel with a shear span to depth ratio of [ ] a,c and marks a transition between flexural failure and shear failure. In addition, the ductile flexural failure of the SC panel with a shear-span-to-depth ratio of [ ] a,c showed that the SC panel had adequate shear transfer capability across the steel-concrete interface."

II. Excerpt from Section 4.3.3 of DPO Panel Report

"The Panel agrees with the submitter in that the ACI 349 requires that ductility should be in every single structural component of the structure. The Panel concludes that the ACI 349 Code requires that all members of the lateral force resisting system should be detailed for seismic ductile performance and that the ductility level is consistent with the special seismic resisting systems contained in Chapter 21 of ACI 349. However, the AP1000 Shield Building satisfies the detailing requirements in Chapter 21 of ACI 349 applicable to the shear walls - [ ] a,c and design stress limits, thus ensuring that the shield building wall as-designed will behave in a ductile manner under seismic loading."

III. Excerpt from Enclosure 5 of DPO Panel Supplementary Report

"The RES report states that while Dr. Vecchio indicated the need for additional testing and tie-bar spacing shorter than [ ] a,c he recognized that ductility requirements in ACI 349 (Chapter 21) relate primarily to frames and not to walls or the Shield Building structure. This expert also understood that ductility detailing requirements in ACI 318 (Ch. 21) relate to structures for which it is allowed to reduce seismic forces by relying on ductile structural response, which is not the case for the AP 1000 Shield Building structure. Recognizing these two aspects, Dr. Vecchio preferred to apply ductility detailing requirements to every region of the Shield Building in the event of unforeseen circumstances in which the loads acting on the structure are of much larger magnitude than anticipated."

"Other RES consultants agreed that given (1) the use of close tie-bar spacing in regions of high out-of-plane shear demand and (2) the low demand-to-capacity ratios presented in the WEC report for those portions of the wall with [ ] a,c tie-bar spacing, it is not appropriate to use the ACI 349 and ACI 318 codes as the basis for insisting on ductility requirements for out-of-plane shear."

IV. Excerpt from the DPO decision document dated February 27, 2015: (Enclosure 7 to DPO panel supplementary report includes a similar statement).

"In describing the SC panel wall with [relatively wide] tie rod spacing as brittle [non-ductile], the Submitter only ever refers to one of the three SC panel tests. The one test, shown as the red curve in the figure on page 12 of the e-mail (see also Panel Report page 31) is for the test that had a shear-span-to-beam-depth ratio of [ ] a,c For the same SC panel tested with a shear-span-to-beam-depth ratio of [ ] a,c a very ductile failure occurred with a ductility ratio of [ ] a,c However, the Submitter fails to mention this test in any discussions."

"To ensure ductile behavior of the SC panel walls everywhere in the SB [shield building], WEC used SC panels with [relatively narrow] [ ] a,c In those [ ] a,c [relatively wide] [ ] a,c Therefore, the SC panels in all regions of the SB will behave in a ductile manner in response to design basis seismic loads."

V. Excerpt from the Statement of Concerns (SOC) prepared by the ad hoc DPO review Panel - "safety concern #3", item (d)(x) and d(xi)

"The NRC staff accepted the SC module element even though when laboratory tested, it failed in a brittle manner and did not meet the acceptance criteria established jointly by the NRC and WEC."

"The NRC staff accepted a non-linear static push over analysis in lieu of the ACI code required testing as a basis for the shield building SC module design. Push over analysis is inappropriate and inadequate. NRC improperly accepted push over analysis neglecting test results. The pushover analysis method is prohibited by building codes for use on an irregular, tall, or safety important building such as the shield building. Therefore, the NRC failed to understand that the nonlinear static pushover analysis is not applicable to the AP1000 shield building."

VI. Excerpt from Section 4.3.2(b) v of the DPO panel report

From the review of the WEC Shield Building design report, expert opinion, and the staff's final safety evaluation report (FSER), the Panel understands that the pushover analysis did not replace the ACI 349 design procedure or requirements; rather it provided an understanding of the beyond design basis response. The pushover analysis only confirmed qualitatively that the shield building stresses, strains and deformations remain small at the design basis loads and that significant yielding in the SC wall does not start until loading levels beyond the SSE [safe shutdown earthquake] and of the order of the RLE [review level earthquake].

The Panel therefore concludes that the design basis of the shield building for SSE is based on elastic response and not on a non-linear static pushover analysis. In accordance with the staff's FSER, it did not accept a non-linear static pushover analysis in lieu of the ACI code required testing as a basis for the shield building SC module design.

c) NRO Staff SER

I. Excerpt from Section 3.8.4.1.1.5 "Shield Building Conclusion"

The staff finds that to resist out-of-plane shear loading, the shield building design uses [ ] [a,c] to ensure that the SC modules will function as a unit. For the regions of the SC wall module with higher out-of-plane shear loads, and where yielding of the SC wall module would be expected to initiate under a combination of tensile forces and out-of-plane bending for seismic loads, the applicant detailed the SC modules with [ ] [a,c] to provide out-of-plane shear ductility. For the regions of the SC wall with low out-of-plane shear demands and [ ], [a,c] the SC wall detailing does not provide out-of-plane shear ductility based on the test results. In these regions, the out-of-plane shear demands calculated by the applicant are low, and the SC wall modules as detailed provide conservative strength demand-to-capacity ratios. Based on: (1) demonstration of conservative strength and adequate cyclic behavior for the SC module with [ ] [a,c]; (2) confirmatory analysis that identified locations of potential SC steel plate yielding; and (3) the analogy with ACI 349, Articles 21.3 and 21.4, which require ductile detailing only where demands are high and plastic hinges are expected to form, the staff finds the applicant's use of [ ] [a,c] to be acceptable.

Regarding out-of-plane shear loading of the SC module with [ ], [a,c] the staff finds that although these specimens failed in a brittle manner, there is significant margin between the failure loads of the two test specimens [ ] [a,c] and the maximum SSE demand of [ ]. [a,c]

II. Excerpt from Section 3.8.4.1.1.3.5 “Design and Testing for Ductility”

The staff finds that testing of SC wall modules with [ ] [a,c] spacing did not demonstrate that the SC wall module is ductile because it did not meet acceptance criteria for ductility as proposed by the applicant.

In addition, the staff finds that the AP1000 shield building design has [ ] [a,c] to ensure that the SC modules will function as a unit. For the regions of the SC wall with higher out-of-plane shear loads, and where yielding of the SC wall would be expected to initiate under a combination of tensile forces and out-of-plane bending for seismic loads in excess of the design-basis loads, the applicant detailed the SC modules with [ ] [a,c] to provide out-of-plane shear ductility. For the regions of the SC wall with low out-of-plane shear demands [tie-bars at [ ], [a,c] and the SC wall detailing does not provide out-of-plane shear ductility. In these regions, the out-of-plane shear demands calculated by the applicant are low and the SC wall modules as detailed provide conservative strength demand to capacity ratios.

The applicant’s approach is to identify, from the results of the analysis for the calculation of member forces and through confirmatory analysis, the locations in the SC structure that are predicted to become plastic hinges (called fuses by the applicant) when subjected to earthquake forces. In the case of the shield building, this requires earthquake forces beyond the design basis seismic loads. Design detailing for the regions in the shield building assumed to be plastic hinge regions conforms to requirements in ACI 349-01, Articles 21.3.3.1-21.3.3.3, which results in shear reinforcing spacing of depth divided by [ ] [a,c] maximum. This detailing is intended to prevent brittle failure modes from pre-empting the ductility of the plastic hinge regions. In regions outside of these assumed plastic hinge locations, the applicant’s design conforms to Article 21.3.3.4, which requires shear reinforcement [ ] [a,c] spaced at no farther apart than half of the depth dimension. In addition, the design for these regions also provides sufficient strength to meet the calculated design demands. Although the ductility detailing requirements in Sections 21.3 and 21.4 of ACI 349 do not apply to the shield building structure, the applicant invoked them for the analogy of the applicant’s design approach to the “capacity design” approach.

The staff evaluated the applicant’s design approach of providing ductility detailing in the regions of high stresses and of providing the strength necessary to meet the design demands in the regions of low demands and finds it to be reasonable. This approach conforms to the approach in ACI 349-01, Articles 21.3 and 21.4 for moment resisting frames, for which ductility design is required by ACI 349, as opposed to structures such as the shield building structure for which ACI 349 does not have ductility provisions or requirements. The staff also finds that the shield building structure, a complex cylindrical shell, distributes loads in a manner that differs from 2D or 3D frames and can be more uncertain. The staff finds that the shield building design provides conservative demand to capacity ratios in the regions of the wall with [ ] [a,c] that can account for those uncertainties. Specifically, the calculated demand to capacity ratios for out of plane shear are for the most part less than or equal to [ ] a,c In addition, the regions of the wall where these demand to capacity ratios are higher than [ ] a,c and as high as about [ ] a,c in a few locations, are small in area and localized.

More specifically, also in Section 2.0 of the September 3, 2010, submittal, the applicant states that the [ ] a,c indicates that for seismic loads greater than the design basis loads, the overturning moment and base shear at the base of the structure cause either tension yielding of the steel plates in the SC portion, or tension yielding of the steel reinforcement in the RC portion of the shield building, depending on the loading combination and direction. In this submittal, the applicant also states that for loads greater than the seismic design basis loads, yielding of the steel faceplates from in-plane shear can occur for certain loading directions. Thus, the ductile failure mechanism for the overall structure is governed by the yielding of steel plates or yielding of steel reinforcement in the RC portion of the structure. The applicant then concluded that for loads greater than the design basis loads, the shield building would develop a ductile failure mechanism with structural fuses in the SC portions located as designed.

The staff finds that the combination of the low demand to capacity ratios for out-of-plane shears in the regions with [ ] a,c spacing with ductility detailing in the regions of high demands provides reasonable assurance of the building safety under the design basis seismic loads by ensuring that the building has structural capacity in reserve, through a combination of structural strength and ductility, for the seismic design basis loads.

III. Excerpt from Page 3-160 (Pushover Analysis)

“The applicant performed nonlinear confirmatory analysis to predict the behavior of the shield building up to and beyond design basis seismic loading and assess the potential for collapse. The applicant used its [ ] a,c model of the nuclear island to perform a nonlinear pushover analysis of the shield building. The model included the shield building and the entire auxiliary building. This finite element model did not impose constraints that would force a mode of deformation of the shield building structure. Using this model, the applicant’s analysis tracked tensile stresses and strains in the steel faceplates, in-plane and out-of-plane shear deformations and stresses, stresses and strains in the [ ], a,c deformations in the connection regions and stresses and strains in the [ ] a,c in the RC wall below the SC wall. The applicant’s analysis explicitly modeled the interaction of the shield building with the roof and walls of the auxiliary building. The applicant’s model also did not exclude the possibility of shear failures. Instead, it considered concrete cracking for out-of-plane loads as well as in-plane loads and the subsequent distribution of forces to the steel reinforcement. Since the applicant’s verification and validation of the model against its own test data did not capture brittle failures, the applicant tracked the possibility of local onset of such brittle shear failures through the use of limiting strains in the [ ] a,c as well as through the combined use of

analysis methods with increasing refinement, that is, the combination of [
] a,c models.

For its analysis, [

]. a,c In addition, the applicant considered various combinations of the directions and intensity of the seismic loads in the two horizontal directions and in the vertical direction. Under these loading conditions and without constraints in the response modes of the structure the applicant calculated the response of the structure to proportionally increasing loads. Proportional increase of the loads is an approximation in a static pushover analysis. As the structure yields and the response becomes increasingly inelastic, there is a potential for redistribution of the loads through the height of the structure that may affect the subsequent response mode of the structure. The results of the applicant's analysis show that significant inelastic behavior of the wall, other than concrete cracking, will not occur at the design basis loads and will only start at loads closer to the review level earthquake (RLE). On this basis, loading conditions that deviate significantly from those used by the applicant are not expected up to the SSE and RLE levels."

<u>DPO Appeal Working Group Perspectives</u>

The NRO staff SER already acknowledged that in the out-of-plane shear test, the specimen with [ ] a,c failed in a non-ductile manner for smaller shear span ratios. Therefore, the submitter's main point regarding the lack of ductility in those areas of the shield building wall with [ ] a,c has already been acknowledged and no value will be added to further discuss the differences between the red (SC specimen test data) and blue (RC specimen test data) curves which have been noted throughout the DPO documentation.

The DPO Appeal working group considered addressing the higher tier and more fundamental point of disagreement, related to the methodology used in the seismic design of the shield building for out-of-plane shear loading, a more effective way to provide a viewpoint. The NRO staff SER accepted WEC's methodology which is similar to a "capacity design" approach. The applicant provided smaller [ ] a,c in those areas of the SC wall where there is a potential for plastic hinge, for beyond design basis seismic load conditions, to ensure ductile failure and demonstrated that in the areas with [ ] a,c spacing the out-of-plane shear demand-to-capacity is low with ample margin. Conversely, the submitter believes that the ACI 349 Code requires ductility in every single structural component of the structure and the tie bar spacing throughout the shield building SC wall must be as such to ensure a ductile failure mode.

The following summarizes the DPO Appeal working group perspectives:

1. The ACI 349 code requires flexural ductility in all members by ensuring tensile reinforcing steel will yield prior to the crushing of concrete in compression. In addition, the ACI 349 requires minimum shear reinforcing steel in members where factored shear force exceeds one-half the shear strength provided by concrete. This minimum shear

reinforcement restrains the growth of inclined cracking. These two basic level of ductility requirements provide assurance that failure without warning is avoided in case of unexpected overload (mainly gravity loads).

For seismic resistant structures, the intent of provisions of Chapter 21 of ACI 349, is that the elements of lateral load resisting system that may experience post-elastic deformation, in the unlikely event of an earthquake beyond the design basis SSE or other unforeseen circumstances, are detailed for ductile performance. The ACI 349 code has provisions for those elements that are not part of the lateral load resisting system to provide assurance that these elements will maintain integrity and can accommodate the anticipated deformations.

2. The capacity design approach is an effective design strategy for earthquake resistant structures and has been discussed in many text books, industry standards/guidance documents and technical papers. In this approach, the designer determines which elements of the structure are going to yield under earthquake excitation. Then, these elements/areas are detailed so that they can sustain yielding without strength degradation. All of the other elements of the structure and their connections are proportioned so that they are strong enough to withstand the maximum forces and deformations that can be delivered to them during the prescribed earthquake. In principle, the elements that are designed to yield act as structural “fuses” to protect other elements of the structure from excessive force.

   As stated in the NRO staff SER, the applicant’s design approach of providing ductility detailing in the regions of high stresses and of providing the strength necessary to meet the design demands with significant margin in the regions of low demands was evaluated and found to be acceptable.

3. One of the key aspects of the capacity design approach is the proper detailing to support the assumptions/results of the structural analysis for identifying the expected yield mechanism. For example, if a non-ductile shear failure is to be avoided, it is necessary to ensure that the shear strength of the element is larger than the shear force associated with the flexural over-strength of that element. The NRO staff SER also stated that the applicant invoked Sections 21.3 and 21.4 of ACI 349 for using the analogy of “capacity design.”

4. ACI 349-01 requires elastic design for safety related structures. Based on a review of the DPO Panel report and the NRO staff SER, it is noted that the AP1000 shield building was designed elastically for SSE loading combinations.

5. For beyond design basis seismic events, the NRO staff SER and the DPO panel report indicate that a pushover analysis was used to study the behavior of the shield building.

6. Although the non-linear static pushover (NSP) analysis is used as a standard tool for seismic assessment of structures, the development and use of the NSP procedure has primarily been based on first-mode dominated symmetrical structures. The NSP analysis does not capture changes in the dynamic characteristics of the structure as yielding and stiffness degradation take place.

7. The structural engineering community and the academia have developed various methodologies to use the NSP analysis for asymmetrical and tall structures.

8. The AP1000 shield building is an asymmetric structure because of its connectivity to the Auxiliary building which makes the [ ]^{a,c} Due to this configuration, [ ]^{a,c} Based on a review of the RES summary evaluation report (Reference 7), dated October 29, 2010 and the staff SER, it is the DPO Appeal working group's understanding that [ ]^{a,c} confirmatory analyses, used in the seismic margin assessment of the AP1000 shield building, were performed using [ ]^{a,c} and models of the entire shield building and nuclear island.

9. Nonlinear response history analysis has traditionally been considered as the most accurate method for estimating the response of a structure to a seismic event. However, due to its complexity and being resource demanding, it is generally used for assessment of special and important structures.

10. The distribution of inertia forces in a NSP analysis will change as portions of the structure yield and dynamic characteristics change. The DPO Appeal working group was unable to locate, in neither the NRO staff SER (Sections 3.8 and 19) nor the DPO panel report, the details of the shield building NSP analysis regarding the distribution of the inertia forces, and verification of the reasonableness of the AP1000 shield building NSP analysis results.

11. The NRO staff SER indicates that for loads greater than the design basis loads the ductile failure mechanism for the overall structure is governed by the yielding of steel plates in the SC portions or yielding of steel reinforcement in the RC portion of the structure.

12. The submitter stated that Dr. Vecchio, in his July 9, 2010 letter, indicated that the load contour plots provided by WEC show that, in general, the moment demand-to-capacity ratios are significantly lower than the shear demand-to-capacity values. In other words, for out-of-plane action, shear demand is more critical than flexural demand; if the loads were increased in fixed proportion, the wall would sustain a brittle shear failure first.

    The NRO staff had already responded to this assertion in Reference 6 stating that (a) NRC consultant, based on demand-to-capacity ratios provided by the applicant, assumed a proportional increase of the loads above the design-basis loads, to conclude that a corresponding proportional increase of out-of-plane shears and moments would lead to shear failure before a flexural failure; and (b) this conclusion by the NRC consultant does not account for tensile forces in the SC faceplates caused by the overturning moments. These tensile forces, when combined with out-of-plane bending moments, [ ]^{a,c} which is consistent with the results from the applicant's analysis.

13. The DPO panel report did not consider the out-of-plane test with shear span/depth ratio of [ ]^{a,c} as a shear test with a ductility ratio of [ ]^{a,c} contrary to the submitter's statement noted in the DPO appeal. The DPO panel's observation was mainly trying to indicate that the effective shear-span-to-depth ratio away from the connection areas (discontinuities) is high; therefore, the shear failure as observed in the test with low span-to-depth ratio is not applicable to areas with [relatively wide] tie rod spacing.

14. The DPO appeal working group was unable to locate a document that the NRC and the applicant (WEC) jointly established and agreed upon ductility criterion for the new SC wall element. The staff SER, Page 3-145, outlines the applicant's ductility acceptance criteria for in-plane and out-of-plane shear tests.

Considering the above, the DPO Appeal working group concludes that the response of the DPO Panel and the NRO staff to the submitter's assertion regarding the out-of-plane shear capacity and ductility of the SC wall was reasonable. However, the DPO Appeal working group recommends the following:

I. The NRO staff should verify that the out-of-plane shear strength of both the SC and RC portions of the shield building wall in the areas of expected plastic hinge - "structural fuse" - is larger than the shear force associated with the flexural over-strength of the SC/RC wall.

Furthermore, considering the fact that (a) the NSP analysis does not capture changes in the dynamic characteristics of the structure as yielding and stiffness degradation take place; and (b) the applicant used the NSP analysis, instead of nonlinear response history analysis, to perform seismic margin assessment and to determine plastic hinge locations/sequence and force redistribution within the AP1000 shield building for a beyond design basis seismic event, the DPO Appeal working group recommends the following:

II. The validity of the inertia force distribution used in the AP1000 shield building nonlinear static pushover analysis should be verified to provide assurance that the shield building dynamic behavior has been adequately captured.

III. The reasonableness of the results of the AP1000 shield building nonlinear static pushover analysis in comparison with the nonlinear response history analysis method should be verified.

<u>REFERENCES</u>

1. Appeal to EDO Regarding NRO Director's Decisions on DPO 2012-002, Safety of AP1000 Shield Building, dated March 31, 2015
2. Differing Professional Opinion Panel Report on DPO-2012-002, Structural Integrity Concerns with the AP1000 Shield Building, dated February 24, 2014 (ML14057A580)
3. Differing Professional Opinion Panel Supplemental Report on DPO-2012-002, Structural Integrity Concerns with the AP1000 Shield Building, dated October 30, 2014 (ML14307A870)
4. Final Safety Evaluation Report Related to Certification of the AP1000 Standard Plant NUREG-1793 Supplement 2 (ML112091879)
5. Memorandum from Mark A. Satorius (Executive Director for Operations) to John Jolicoeur (Executive Technical Assistant), Differing Professional Opinions Appeal on AP 1000 Shield Building Issues (DP0-2012-002), dated September 8, 2015 (ML15243A498)

6. Staff Response to Dissenting View on the Safety Evaluation Report for the Design of the AP1000 Shield Building (SRP Section 3.8.4), dated November 8, 2010 (ML103020239)
7. Summary Evaluation Report - Enhanced Shield Building Structure, Westinghouse Electric Company Report APP-1200-S3R-003. Revision 2 AP1000 Design Control Document, Revision 17; Office of Nuclear Regulatory Research, USNRC, October 29, 2010 (ML103080129)

This enclosure provides the DPO Appeal working group perspectives on the in-plane shear strength of the shield building SC wall based on a review of information included in the submitter's DPO appeal (Reference 1), the DPO Panel Report (Reference 2), the DPO Panel supplementary report (Reference 3), and the NRO staff SER (Reference 4).

As directed in the EDO memorandum (Reference 5), in light of the significant work already conducted in this case, the limited scope and primary focus of the DPO Appeal working group was to apply the reasonableness test, as opposed to initiating an independent re-review, to issues raised in the DPO appeal against the responses provided in the DPO Panel reports and the NRO Office Director's findings to determine whether the issues were addressed sufficiently. Therefore, the perspectives of the DPO Appeal working group should not be construed as a peer review of the AP1000 shield building design.

Considering the above, the following excerpts from References 1 through 4 provide the background of the main technical issues regarding the in-plane shear strength of the shield building wall.

a) Submitter Assertion (DPO Appeal, Reference 1)

   I. The DPO panel's evaluation on the shear strength of the AP1000 shield building relied solely on Mr. Loring Wyllie's [DPO Panel consultant] opinion and judgment and considered the use of [ ]^{a,c} conservative, instead of the ACI Code's limit of $8\sqrt{fc'}$, which is the design basis of the AP1000 shield building.

   II. The shear stress due to review level earthquake (RLE) exceeds the value of [ ]^{a,c} contradicting the DPO Panel Report statement that the AP1000 shield building wall has sufficient shear strength to resist RLE satisfying NRC seismic margin requirements.

   III. There are modeling deficiencies (accidental torsion, P-delta effect, and irregular zigzag connection between the SC and RC wall) of the AP1000 shield building that could underestimate the magnitude of shear stress in the wall.

b) DPO Panel Report (Reference 2) and DPO Panel Supplementary Report (Reference 3)

   I. Excerpt from Section 4.1.2.4 of DPO Panel Report

      The Panel's expert, Mr. Wyllie concluded that the in-plane shear capacity of the SC wall should have been calculated considering the [ ]^{a,c} For in-plane shear, the ACI 349 equations are probably not directly applicable as the [ ]^{a,c} provide for more shear capacity than a normal reinforced concrete wall. Using ACI 349, shear capacity would be limited to $8\sqrt{fc'}$ by Chapter 21. WEC used a capacity of [ ]^{a,c} for shear. The true in-plane shear strength is much greater and dependent on [ ]^{a,c} provide bond of the steel plates to the concrete core.

"The Panel notes that for SSE loads, WEC conservatively ignored the contribution of the [ ] [a,c] and used the in-plane shear design capacity of the shield building wall as [ ] [a,c] based on the [ ] [a,c] WEC's estimate of the expected in-plane shear capacity based on Japanese tests that considers only the inner and outer steel plates is [ ] [a,c]

The computed in-plane shear capacity of [ ] [a,c] in Appendix L of the Shield Building Report (page L-93) was based on empirical equations supported by test data. The Panel is of the view that it is prudent to reduce the estimated in-plane shear capacity to account for uncertainties. Due to the potential uncertainties in the test program and the uniqueness of the SC structure, the Panel conservatively reduced the in-plane shear capacity by half [ ] [a,c]. The computed in-plane shear force demand in the shield building wall due to SSE [

] [a,c] is lower than the reduced in-plane shear capacity of [ ] [a,c] The in-plane shear capacity will envelope the RLE demand. This consideration also leads to a more realistic SSE design margin of [ ] [a,c] for in-plane shear force.

II. Excerpt from Section 4.1.4 of DPO Panel Report

"Based on the simplified and conservative assumptions for the HCLPF estimate for the shield building wall in-plane shear force, the Panel concludes that the HCLPF exceeds the review level earthquake (RLE) of 0.5g and therefore, the thickness of concrete in the shield wall is sufficient to resist in-plane shear generated by the review level earthquake. The Panel further concludes that the shield building wall satisfies the NRC seismic margin requirements."

III. Excerpt from Enclosure 5 to DPO Panel Supplementary Report

"In his email and attachment, the submitter cites a 1999 American Concrete Institute (ACI) report (ACI 445R-99) and National Earthquake Hazards Reduction Program (NEHRP) guidance which indicate in-plane shear strengths less than $10\sqrt{fc'}$ under certain conditions. Staff review of the information finds that while it contains useful research and insights, the information has not reached the level of consensus to be codified in the latest building codes, which incorporate the state-of-the-art knowledge on the subject. This is evidenced by the observation that both ACI 349-01 and ACI 318-11, published after the ACI 445R report, contain shear wall provisions with a limit value of $10\sqrt{fc'}$. In addition, the actual capacity of the AP1000 Shield Building's steel concrete composite (SC) walls was shown to be significantly greater than the ACI code capacity limits (see Figure 4 below). This is because the steel plates can carry large tension and compressive forces needed to resist in-plane shear. As such, the submitter's concerns related to the in-plane shear capacity of the AP1000 Shield Building's SC walls are of low safety significance."

"Staff notes that the issue of in-plane shear capacity has been extensively discussed in the 2010 SB non-concurrence (NC) (ADAMS Accession No. ML103370648), the staff response to the NC (ADAMS Accession No. ML103020239), and the DPO Panel Report). This is not a new issue and the DPO panel addressed it thoroughly in its report."

IV. Excerpt from Section 4.3.2(a)(i) of DPO Panel Report

"The cylindrical shell of the shield building (SB) intersects the rectangular walls and floors of the auxiliary building (AB). The intersection of the AB with the SB forms a continuous series of vertical and circumferential lines. These intersection locations where the SB is integrally connected to the AB are also the locations where the SC portion of the SB cylindrical wall transitions to RC and are referred to by the Submitter as the "irregular zigzag or step shape" boundary condition."

"The "torsional irregularity" as a result of the zig-zag shape of the boundary connecting the SC and RC portions of the SB wall is created by the difference in stiffness between the SC and RC portions of the wall and the fact that the zig-zag boundary [ ] [a,c]

"….the increased stiffness of the SC portion over the RC portion was calculated by WEC [ ] [a,c] used to calculate the seismic demand on the SB."

"It is the Panel's expert judgment that the WEC finite element models have been constructed to adequately replicate the physical, geometric and material properties of the AB and SB structures, including the connectivity of the SB to the AB along the zig-zag boundary and the difference in stiffness between the SC and RC portions of the SB. The Panel, therefore, concludes that the effects of the dynamic interaction between the AB and the SB, and the effects of inherent torsion due to the difference in stiffness between the SC and RC portions are appropriately reflected in the seismic analysis results for the SB wall."

"To account for accidental torsion NUREG-0800, Section 3.7.2 requires that an additional eccentricity of 5% percent of the maximum building dimension be assumed for both horizontal directions separately for each floor elevation. The SRP also states that, "An acceptable alternative, if properly justified, is the use of static factors to account for torsional accelerations in the seismic design of Category I structures." ACI Codes 318 and 349 do not specifically require consideration of accidental torsion."

"The [ ] a,c model used by WEC to perform the seismic analysis is a [ ] a,c model of the AP1000 nuclear island. In order to obtain the proper boundary conditions between the shield building cylinder and the other portions of the nuclear island, the shield building is modeled with the nuclear island that includes the auxiliary building and containment internal structure (CIS)."

c) NRO Staff SER

Excerpts from pages 3-147, 3-148 and 3-149:

"In summary, the staff finds that the purpose of shear tests is to establish the minimum shear reinforcement [ ] a,c to the SC module so that it can function as a unit to resist both out-of-plane and in-plane shear forces, provide sufficient ductility (energy absorption/dissipation capability) for seismic-induced energy, and provide sufficient stiffness for the shield building to meet the allowable building drift limit. The staff finds that the tests were an acceptable basis to establish this minimum."

"The staff's review of the test plan for the in-plane shear test (Section 7.12) finds that the test model and test set-up boundary conditions [ ], a,c as shown in Figures 7.12-1 to 7.12-5, may provide additional resistance and can lead to an over-estimation of the actual strength of the SC wall module. The applicant had to terminate the test after [ ] a,c due to laboratory safety constraints and, therefore, could not complete the ductility test."

"The staff finds that although there were concerns regarding the test setup at Purdue, the test results indicate that the design for the in-plane shear strength criteria used [ ] a,c is adequate."

"The staff reviewed the Ozaki paper, and found that the test was properly conducted and credible. In SER Table 3.8-1, staff performed a review of the Ozaki, et al. paper to compare a few key parameters of the AP1000 design and the S4-00NN specimen. Based on this comparison, and the good agreement of SC parameters, the staff finds the applicant's use of the test data to demonstrate ductility of the SC wall to be appropriate."

"For the in-plane shear test, the staff finds that the test results indicate that the design for the ACI 349 the in-plane shear strength criteria used, [ ] a,c is adequate. The test results were inconclusive with respect to measurable ductility. However, cyclic ductility tests performed in Japan (documented in the Ozaki paper) indicate that the wall will exhibit ductile behavior under cyclic in-plane shear. On these bases, the staff concludes that the SC wall will provide adequate strength, stiffness, and ductility under design-basis (or SSE) seismic loads."

DPO Appeal Working Group Perspectives:

The following summarizes DPO Appeal working group perspectives:

1) The ACI 349 code has provisions in Chapters 11 and 21 relative to the in-plane shear capacity of the RC walls. The upper bound limit of the in-plane shear capacity for the RC walls is $10\sqrt{fc'}$ according to Section 11.10 of ACI 349. However, the nominal and the upper bound limit of the in-plane shear capacity of the RC portion of the shield building wall should be calculated in accordance with Chapter 21 of ACI 349.

2) The use of [ ]^{a,c} of the SC wall has been supported by the in-plane shear tests. The NRO staff SER found the experimental results obtained in the United States and Japan, to predict the in-plane shear behavior/capacity of the SC wall, acceptable and applicable to the AP1000 shield building SC wall.

3) The DPO Panel Report mainly addressed the in-plane shear capacity of the SC wall and noted that WEC conservatively ignored the contribution of the [ ]^{a,c} and used the in-plane shear design capacity of the shield building wall as [ ]^{a,c} based on the [ ]^{a,c} alone. The DPO Panel Report further noted that the computed in-plane shear capacity of [ ]^{a,c} was based on empirical equations supported by test data for SC modules. [ ]^{a,c}

   In addition, according to Appendix N9 “Steel-Plate Composite (SC) Walls” of AISC N690 (Reference 6), currently under review by the NRC staff/contractor for endorsement, the in-plane shear strength of the SC wall is calculated based on the strength of the face plates. The calculation based on Formula A-N9-19 of Appendix N9 of AISC N690-12 (Reference 6) yields the in-plane shear strength of the shield building SC wall as [ ]^{a,c} which is in-line with the in-plane shear capacity cited in the DPO Panel report and it supports the notion that calculating the in-plane shear capacity of the AP1000 SC wall based on [ ]^{a,c} is conservative.

Considering the above, the DPO Appeal working group concludes that the response in the DPO Panel Report to the submitter’s assertion regarding the in-plane shear capacity of the SC wall was reasonable.

The DPO appeal also stated that there are modeling deficiencies (accidental torsion, P-delta effect, and irregular zigzag connection between the SC and RC wall) of the AP1000 shield building that could underestimate the magnitude of shear stress in the wall. This assertion was also included in the original DPO and responded in Section 4.3.2 (a)(i), (ii) and (iii) of the DPO Panel report and the DPO Panel supplementary report. In this regard, the DPO Appeal working group concludes that the submitter did not provide any new information regarding the modeling deficiencies and the DPO Panel report, including the DPO supplementary report, and the NRO staff have adequately responded to these assertions.

REFERENCES:

1. Appeal to EDO Regarding NRO Director's Decisions on DPO 2012-002, Safety of AP1000 Shield Building, dated March 31, 2015
2. Differing Professional Opinion Panel Report on DPO-2012-002, Structural Integrity Concerns with the AP1000 Shield Building, dated February 24, 2014 (ML14057A580)
3. Differing Professional Opinion Panel Supplemental Report on DPO-2012-002, Structural Integrity Concerns with the AP1000 Shield Building, dated October 30, 2014 (ML14307A870)
4. Final Safety Evaluation Report Related to Certification of the AP1000 Standard Plant NUREG-1793 Supplement 2 (ML112091879)
5. Memorandum from Mark A. Satorius (Executive Director for Operations) to John Jolicoeur (Executive Technical Assistant), Differing Professional Opinions Appeal on AP 1000 Shield Building Issues (DP0-2012-002), dated September 8, 2015 (ML15243A498)
6. ANSI/AISC N690-12, “Specification for Safety-Related Steel Structures for Nuclear Facilities including Supplement No. 1”

This enclosure provides the DPO Appeal working group's perspectives of the concern related to punching-shear failure of the AP1000 shield building SC wall due to a beyond design-basis aircraft impact. These perspectives are based on a review of information included in the submitter's DPO appeal (Reference 1), NRO Office Director's decision (Reference 2), DPO Panel report (Reference 3), the DPO Panel supplementary report (Reference 4), the NRO Director's statement of views (SOVs) on the DPO Appeal (References 5, 6), the NRO Inspection Report for Aircraft Impact Assessment (Reference 9), the ACRS Report on the Safety Aspects of the Aircraft Impact Assessment for the Westinghouse Electric Company AP-1000 Design Certification Amendment Application (Reference 21), and additional references listed under the "References" section of this enclosure.

As directed in the EDO memorandum (Reference 7), in light of the significant work already conducted in this case, the limited scope and primary focus of this review was to apply the reasonableness test, as opposed to initiating an independent re-review, to issues raised in the DPO appeal against the responses provided in the DPO Panel reports and the NRO Office Director's findings to determine whether the issues were addressed sufficiently. Therefore, the perspectives of the DPO Appeal working group should not be construed as a peer review of the AP1000 shield building design.

Considering the above, the following information summarized from References 1 through 6, 9, 21, and 30, provide the background of the main technical issues raised by the submitter related to punching-shear failure of the AP1000 shield building SC wall due to an aircraft impact.

a) Submitter's Assertion (DPO Appeal, Reference 1)

The design of the AP1000 shield building SC wall is very strong, with great amount of energy absorption capability, in resisting bending (flexure), but very weak with small energy absorption capability in shear, based on the [ ] a,c (indicated by the "red curve" in a figure in DPO 2012-002). The actual local failure mode of the wall, as a result of aircraft missile strike, should be punching shear instead of [ ] a,c as was evaluated and which controlled the shield building wall thickness for the design-basis hurricane-generated automobile missile. As such, the shield building SC wall must be much thicker to adequately resist local aircraft impact loading in a punching-shear failure mode.

The submitter asserted the following arguments in support of the above position:

I. Citing information on slide 69 of the presentation to the NRO Office Director and DPO Panel on September 8, 2014 (Reference 8), with the exception of the AP1000 shield building, the required concrete wall thickness for not being punched through by aircraft in the other designs or analyses listed there was about 6 feet. The punching shear strength for a concrete slab, footing or wall is related to the square power of thickness and, [ ] a,c 6-ft thick wall [ ] a,c

[ ] a,c

II. The AP1000 shield building wall is neither simply supported nor fixed end supported in the circumferential direction, as considered in Dr Rashid's [DPO Panel consultant] arch-shaped mathematical model to demonstrate the role of curvature in providing superior impact resistance of the shield building wall (versus flat SC plates), with its fixity being some value in between. Therefore, the DPO Panel's explanation that the arch-action from curvature, and support conditions, of the shield building were responsible for its great resistance to perforation by punching shear under aircraft impact is not convincing.

III. The DPO Panel consultant's aircraft impact analyses results from the computer codes indicated a bending failure mode, suggesting that the shear failure mode (the red curve) had not been, or had been incorrectly, coded into the computer analytical models.

b) Office Director's Decision (Reference 2), DPO Panel Report (Reference 3), DPO Panel Supplementary Report (Reference 4), and Statement of Views on DPO Appeal (References 5, 6)

The NRO Office Director's (OD) decision memo addressed the issues raised by the submitter, related to punching-shear failure under aircraft impact, as part of Safety Concern 2 in Section 3.3 and specifically in Section 3.3.2. The OD's decision agreed with the DPO Panel's conclusion that the AP1000 shield building is capable of withstanding an aircraft impact, satisfies NRC's aircraft impact assessment requirements, and there is reasonable assurance that the design protects public health and safety (Reference 2, Sections 3.3.1 and 3.3.4).

The DPO Panel addressed the submitter's above concerns in Section 4.2, and specifically in subsections 4.2.3.1, 4.2.3.2, 4.2.3.3, 4.2.3.4, 4.2.3.5.2, and 4.2.3.5.3 of the DPO Panel Report. Additionally, the staff's and DPO Panel's supplementary evaluation of AIA benchmarking, the submitter's comparison of AIA results for the AP1000 SB and another design (EPR), and the influence of arch-action from curvature effects of the shield building are documented in Enclosures 4, 5, and 6, respectively, of the DPO Panel Supplemental Report. The NRO OD's statement of views (Reference 5) regarding the DPO appeal concluded that the appeal does not provide any new information or new interpretation of existing information that changes the positions described in the decision. The basis for this conclusion was documented in a summary of a meeting held on June 17, 2015 (Reference 6). Excerpts of how these documents addressed the concerns raised in the DPO appeal are included below.

Excerpts from the basis (Reference 6) for the OD's SOVs (Reference 5) on the DPO Appeal:

> Automobile impact is a design basis event which is analyzed using standardized procedures [for punching shear] in structural design standards. This approach

> was used by the applicant and NRC staff for assessment and review of the design basis tornado missile impact on the AP1000 nuclear island.
>
> For assessment of the more complex dynamic impact loads from beyond design basis aircraft impacts, the applicant and NRC staff used the guidance of Regulatory Guide (RG) 1.217, "Guidance for the Assessment of Beyond-Design-Basis Aircraft Impacts." This guide describes usage of three-dimensional non-linear finite element analyses to account for the various mechanisms and interactions that resist the impact loads.
>
> The RG 1.217 approach was used by Westinghouse for the AP1000 shield building analysis, and explicitly addressed punching shear. The model used by Westinghouse was sufficiently detailed to determine local and global effects. The NRC staff reviewed the Westinghouse analysis and concluded that it met regulatory requirements. This analysis and two other [independent AIA] analyses described in the Panel's reports [one by the DPO Panel consultant (Dr Joe Rashid) which focused on local effects, and one by NRC Office of Nuclear Regulatory Research (NRC/RES) which focused on global effects] demonstrate that punching shear was not a controlling failure mode.
>
> ….The submitter compares the AP1000 [shield building] to the EPR shield building. However, the two structures are fabricated differently, with AP1000 using SC panels while EPR uses conventional reinforced concrete.

Excerpt from Enclosure 4 of DPO Panel Supplemental Report of the NRO/DE evaluation of AIA Benchmarking:

> Between September 27 and October 1, 2010, staff performed an inspection of the WEC [Westinghouse Electric Corporation] AIA and specifically focused on the issue of concern (i.e., analytical model validation). To support the staff, a structural engineering expert from Sandia National Laboratory who was familiar with the operating fleet's vulnerability assessments helped to review the AIA of the Shield Building. The inspection report (ADAMS Accession No. ML102980583) describes the staff's review of the AIA to ensure consistency with subsection 2.4.1(4) of NEI 07-13 [regarding benchmarking of computer models for AIA] as follows:
>
> > The NRC inspection team reviewed the WEC AP1000 AIA structural damage assessment including design inputs, analysis parameters and assumptions, computer codes, method used for structural analyses and results. Specifically, the NRC inspection team reviewed the LS-DYNA computer code used in the structural analysis for the AP1000 AIA to determine if the applicant had adequately validated and verified the code for the applicable class of problems assessed and had adequately documented the validation and verification. DG-1176, in Section 2.4.1 of NEI 07-13 states that "new design features may be subject to failure

modes that are outside of their existing experience base, and may require experimentally-verified analytical evaluation" or benchmarking.

The AP1000 Shield Building makes use of steel concrete composite construction for which there is greater uncertainty with respect to impact behavior compared to reinforced concrete. WEC performed benchmarking of the LS-DYNA analysis code on steel concrete structures using the Winfrith concrete model. The NRC inspection team reviewed the benchmarking process and the technical justification and verified it to be accurate and complete.

During subsequent inspections (in May 2011 through August 2011; see ADAMS Accession No. ML112650748), the staff re-examined WEC's benchmarking and verified that the [

] a,c

Excerpt from Enclosure 4 of DPO Panel Supplemental Report of the DPO Panel's Supplementary Assessment of AIA Benchmarking

The purpose of analytical model validation and benchmarking is to provide assurance that the computer code can adequately model the SB wall's structural behavior to simulate the response due to aircraft impact. The staff's inspections of WEC's model validation and benchmarking provide the Panel with additional assurance that the computer code can adequately model the SB wall's structural behavior to simulate the response due to aircraft impact. To further assure that the WEC analytical model validation and benchmarking efforts resulted in a quality AIA, the NRC staff, using the same computer code as WEC, performed an independent detailed AIA, and confirmed the WEC result that the aircraft does not result in perforation of the SB wall. In addition, the DPO Panel's aircraft impact expert, using a different computer code, confirmed the same result (See DPO Panel Report pages 34 and 35.).

Excerpts from Enclosure 5 of DPO Panel Supplemental Report of the staff's evaluation and DPO Panel's supplementary evaluation:

The submitter's email [dated May 1, 2014] makes a comparison of AIA results for the AP1000 SB and another nuclear power plant design (EPR). This issue is also discussed in the submitter's DPO and addressed in the DPO panel report (in Section 4.2.3). The submitter's concerns of inadequate wall thickness, based on other certified designs (that have been or are under review), have been raised before by the submitter. The staff position is that this approach is an oversimplification that neglects various significant aspects that influence the behavior of a wall subject to impact load, such as reinforcement ratio, wall span/radius, and curvature. The AP1000 Shield Building wall thickness has been verified by three independent AIAs to be adequate.

…The DPO Panel agrees with the staff's evaluation of the Submitter's email of May 1, 2014 contained in Enclosure 5.

Excerpt from Section 4.2.3.1 of DPO Panel Report:

The SC panel in the AP1000 [shield building] forms a curved cylindrical shell. …... It is the curvature of the AP1000 shell wall, which was not incorporated in the Purdue tests, that significantly reduces the shear stresses in the SC wall under external loading, enhances ductile behavior and allows large amounts of energy to be absorbed. Therefore, in the context of aircraft impact, the test results for a flat SC panel cannot be directly compared to a curved SC panel which is part of a continuous shell.

Excerpts from Section 4.2.3.2 of DPO Panel Report:

Due to their curvature, thin shells offer important advantages over flat plates. Shell structures permit substantial savings of material (i.e., reduced thickness) compared with designs using flat concrete walls. This *results, "from their ability to translate applied loads into 'membrane' thrusts and shears acting in a plane tangent to the surface at any point. By this means bending and twisting moments, and shears transverse to the surface, are reduced or eliminated" [4.2(g)].* This type of structural behavior, which is unique to shells, is often referred to as "arching action." It is the arching action of the cylindrical shield building shell wall, more than any other single feature, that explains the fact that the [ ] [a,c] thick shell of the AP1000 shield building wall can resist aircraft impact loads without being perforated, while the 3'-6" thick RC flat wall of the ESBWR reactor building, the 4'-0" thick RC flat wall of the STP ABWR reactor building, and the 4'-0" thick SC wall analyzed by Mullapudi, are all perforated by the aircraft impact loading.

……In his report [4.2(h)], Dr. Rashid concludes: *"The analysis results... for the flat Panel... show... perforation begins at 0.176 seconds when pieces of the Panel start to break up and fly away." "The analysis of the curved Panel... progressed sufficiently to show that the curved Panel will not perforate." "The purpose of the analysis is not to check Mullapudi's results, but rather to quantify the effects of a curved-wall design relative to a flat-wall design of similar design features. Clearly, the results of Mullapudi analysis cannot be used to represent the design capacity of the AP1000, because it totally ignores the most important features of the design..."* - the arching action effect.

Excerpt from Section 4.2.3.5.3 of DPO Panel Report:

> The same proven techniques used to develop RC finite element models are used to develop the finite element model of the SC module. Dr. Rashid has spent over 40 years developing concrete constitutive laws and developing composite finite element models. Again, the techniques used to construct composite models of the SC module are the same as those used to construct the composite models of RC. The constitutive laws of all components: the concrete, the steel, the Nelson studs, the shear ties, have been well known and continuously improved through decades of testing and analytical research, as are the techniques for connecting them to form the composite component.

Excerpt from Section 4.2.5.1 of DPO Panel Report:

> In [DPO Panel Report] Section 4.2.3.2 the Panel's expert, Dr. Rashid, demonstrated the significant effect that shell curvature has on the ability of the SC panel to resist aircraft impact. It is this curvature, which is the reason why the SC panel of the SB can survive an aircraft impact without perforation while thicker SC flat panel and RC walls are perforated by the aircraft.
>
> Based on the Panel's review of the aircraft impact analysis performed by Dr. Rashid, the Panel concurs with his conclusion that*, "the AP1000 SC shield building is competent to stop a large commercial aircraft without perforation of the wall or threatening the integrity of the containment."*

c) NRO Inspection Report No. 05200006/2010-203 (Reference 9):

Excerpt from "Executive Summary:"

> …. This AIA inspection was performed to verify that the WEC AP1000 AIA complies with the requirements of 10 CFR 50.150 and to ensure consistency with the industry guidance documented in Nuclear Energy Institute (NEI) 07-13, "Methodology for Performing Aircraft Impact Assessments for New Plant Designs," issued May 2009. NEI 07-13 has been endorsed by the NRC in Draft Regulatory Guide 1176 (DG-1176) "Guidance for the Assessment of Beyond-Design-Basis Aircraft Impacts," as one means of performing an AIA acceptable to the NRC. …

Excerpts from Section 3.b.3 "Containment structure and spent fuel pool specific impact assessment":

> The applicant properly accounted for the effects of local impact loading by performing detailed finite element analyses using wall panels and a projectile

whose parameters matched those supplied by the NRC. These analyses were performed using benchmarked analysis methods.

……The applicant used the Winfrith concrete damage model consisting of material properties and equations used to model the nonlinear behavior of concrete materials used in the analyses. The various steel components, including reinforcements, were modeled with appropriate elasto-plasticity models.

Excerpts from Section 3.a "Inspection Scope of partial list of documents reviewed by staff related to computer code benchmarking for "Structural Damage Assessment:"

6. APP-1000-S2C-041, "LS-DYNA Benchmarking for OOP Shear Test for SC Beam with a/d=3.5," Revision 0, dated September 27, 2010.

11. SMIRT-18-J05-1, "Investigation on Impact Resistance of Steel Plate Reinforced Concrete Barriers against Aircraft Impact," Part 1: "Test Program and Results," dated August 12, 2005

Excerpts from Section 3.b.2 "General Structural Analysis":

The AP1000 Shield Building makes use of steel concrete composite construction for which there is greater uncertainty with respect to impact behavior compared to reinforced concrete. WEC performed benchmarking of the LS-DYNA analysis code on steel concrete structures using the Winfrith concrete model. The NRC inspection team reviewed the benchmarking process and the technical justification and verified it to be accurate and complete.

The NRC inspection team reviewed the assumptions used in the structural damage analyses. For the purpose of validating predicted structural damage to the Shield Building, the applicant performed an analysis of an impact experiment involving steel concrete (SC) panels and a deformable projectile. The applicant's comparison of predicted and measured results agreed reasonably well. The NRC inspection team verified that the applicant used sufficient modeling and meshing refinement in the structural damage analyses.

d) Advisory Committee on Reactor Safeguards (ACRS) Report on the Safety Aspects of the Aircraft Impact Assessment for the Westinghouse AP-1000 Design Certification Amendment Application (Reference 21)

Excerpt from "Conclusion and Recommendation" Section:

The WEC AIA for the design described in the AP1000 DCA application, as modified to resolve NRC inspection findings, complies with the requirements of 10 CFR 50.150.

Excerpts from "Discussion" Section:

Related to <u>AIA for PCS tank and debris impact:</u>

The AP1000 shield building includes a 32 ft. diameter opening in the conical roof which is an essential feature of the passive containment cooling design. This opening is surrounded by the Passive Containment Cooling System water storage tank. During our November 2-3, 2010, subcommittee meeting, issues arose concerning the potential for significant aircraft impact debris to pass through the opening and impact the steel containment vessel. WEC conducted appropriate analyses, which we reviewed during our November 17-19, 2010, subcommittee meeting. Using realistic assumptions for the impact locations of concern, these analyses demonstrated that no significant debris would impact the steel Containment Vessel (CV). In addition, WEC performed a more conservative analysis in which a large mass consisting of debris and the shield plate, was assumed to fall on the steel CV. This impact resulted in only a relatively small amount of plastic deformation and no penetration of the CV.

Related to <u>Benchmarking of computer code used for the AIA:</u>

Our December 13, 2010, letter concerning the AP1000 DCA application describes the SC design, including the addition of tie bars between opposite faceplates of the SC modules. The spacing of these tie bars is smaller in areas of higher, out-of-plane, design basis shear demands - i.e., near discontinuities and connections - than it is in the majority of the shield building wall structure where these demands are lower. Aircraft impacts, unlike design basis events, can impart high out-of-plane shear demands in regions of the shield building wall with greater tie bar spacing. As discussed in our letter of December 13, 2010, these areas can fail in a non-ductile manner under such loads. In order to assure acceptable realism in the analyses, it must be demonstrated that the finite element models used in the **AIA** adequately describe this non-ductile behavior under high out-of-plane shear loads. WEC provided comparisons of the predictions of the LS-DYNA model with an experiment on a beam representing a SC structure with greater tie bar spacing under high out-of-plane shear loads. The load-deformation behavior predicted by the model agreed well with the results of the experiment; the comparison adequately supports the use of the model for these analyses.

In addition to the possibility of global structural failure, there is also a potential for local failure due to penetration by hard objects such as an engine or landing gear. The **AIA** analysis included comparisons of the predictions of the LS-DYNA model with penetration tests conducted in Japan on SC structures. The predictions show adequate agreement with the tests. Although the geometry of the specimens in these tests differs from that of the shield building, the comparisons support the use of the model to predict local failures associated with aircraft impact.

e) Closure Memo of Follow-up Action from the Decision Regarding DPO 2012-002 with focus on the Passive Containment Cooling System Tank and Postulated Debris (Reference 30)

Based on the recommendation in the DPO Panel Supplemental Report, as one of three follow-up actions, the NRO Office Director's Decision directed the NRO staff to perform an inspection of WEC's aircraft impact assessment documentation, focusing on the PCS tank and any postulated debris impact on the steel containment vessel. This follow-up action was completed by NRO. The NRO staff conducted the inspection on July 27-31, 2015, December 9-10, 2015, and February 11, 2016, and the results of the inspection are documented in NRC Inspection Report No. 9900404/2015-203 dated April 16, 2016 (ADAMS Accession No. ML16099A049), and closure of the follow-up action is documented in a memo to the NRO Office Director dated June 24, 2016 (ADAMS Accession No. ML16146A662). A partial excerpt from the closure memo documented that:

> The staff verified that the final modeling and analyses were reasonable and realistic and agreed with WEC's conclusion that the inner wall of the PCS tank would remain structurally intact so that no significant debris would be expected to impact the shield plate and subsequently the steel containment vessel. In addition, the staff reviewed: 1) the analysis of the shield platform assembly impacted by a large piece of debris falling from above, 2) the analysis of the steel containment [vessel] impacted by the shield platform assembly with a large amount of accumulated debris, and 3) the analysis of shield building roof impacted by a large commercial aircraft. The staff found the assumptions in these analyses were realistic and the results acceptable, and agreed with WEC's conclusion that the containment pressure boundary would not be breached.

<u>DPO Appeal Working Group Perspectives</u>

The DPO Appeal working group met with cognizant staff in RES on December 5, and 17, 2015; and with the DPO Panel on February 8, 2016, to gain insights into their independent aircraft impact analyses of the AP1000 shield building.

The submitter's claim that the AP1000 shield building SC wall will fail in the punching-shear failure mode is based on the “brittle [non-ductile]” shear test failure, indicated by the “red curve” in a figure in DPO-2012-002, observed in the monotonic static out-of-plane load test of a large-scale SC-wall beam test specimen with shear span-to-depth ratio of [ ] [a,c] As documented in Section 7.7 of the Design Report for the AP1000 Enhanced Shield Building (Reference 10), this test and two others with different shear span-to-depth ratios satisfied its objective of demonstrating that the out-of-plane shear strength of the AP1000 SC wall can be estimated using ACI 349 code provisions for reinforced concrete beams. [

] [a,c] It should be noted that there was no indication of tear or fracture of either of the faceplates when the test was concluded. Further, it should be noted that punching-shear (two-way shear) behavior, which is the submitter's issue of concern, is not dependent on shear span-to-depth ratio and the "red curve" indicates data from an out-of-plane shear (one-way shear) test and is not representative of punching-shear behavior. The submitter's claim that the [ ]^{a,c} 6 ft thick [ ]^{a,c} does not factor the presence of the two solid steel faceplates in the SC wall (or recognize that there is a difference in structural behavior between RC and SC structure), and is considered a simplification of the ACI Code punching shear strength equation because it does not consider that the critical perimeter is defined by the loaded area with each side increased by the thickness, and not by the thickness alone.

As explained in Section 4.2.3.5.3 of the DPO Panel Report, aircraft impacts are beyond design-basis events and are beyond the scope of and not evaluated to the requirements of construction codes such as ACI 349 or ACI 318 used to design the structure for design-basis loads and load combinations. As indicated in Section 19.5 of NUREG-0800 Standard Review Plan (SRP), aircraft impact assessments (AIA) of new reactor designs to meet the requirements of 10 CFR 50.150 are recommended to be performed in accordance with Regulatory Guide (RG) 1.217 August 2011 *(Draft issued as DG-1176, dated July 2009, was used by Westinghouse for the AP1000 AIA),* "Guidance for the Assessment of Beyond-Design-Basis Aircraft Impacts." This RG endorses the guidance in Nuclear Energy Institute (NEI) 07-13, "Methodology for Performing Aircraft Impact Assessments for New Plant Designs," as an acceptable method for use in satisfying the NRC requirements in 10 CFR 50.150(a) regarding the assessment of aircraft impacts for new nuclear power reactors.

For realistic analysis, it is not appropriate to compare traditional and conservative ACI code-based punching-shear (two-way shear) evaluation methods, for design of reinforced concrete slabs and footings (where the static punching-shear force is easily calculated from design loads) that was conservatively used by the applicant for "design-basis" impact loading (e.g., hurricane- or tornado-generated automobile missile) to the more complex realistic and best-estimate analyses methodologies that were used by the applicant, the DPO Panel's consultant, and NRC/RES to analyze the effects of short-duration beyond-design-basis aircraft impact loading. These impact analysis methodologies use non-linear dynamic finite element analyses of the shield building in its structural context as a cylindrical shell structure. Because of the complexity involved in transforming a dynamic impact energy to an impact force that varies with time during the duration of impact, these methods use the NRC-specified impact force time-history of the aircraft missile (generally for global analysis) or apply the NRC-specified missile impact parameters, or equivalent, as an initial velocity problem (generally for local impact loading) to define the dynamic impact loading. Further, it is important to clarify that there is a difference in the definition of "failure criterion" for design-basis loads and beyond-design-basis aircraft impact loading. For beyond-design-basis aircraft impact events, consistent with the guidance in RG 1.217 (Reference 13) and NEI 07-13 (Reference 14), the selection of dynamic strength properties including strain-rate effects and non-linear strain-based failure criteria representative of realistic best-estimate material behavior beyond yield strain is appropriate, and the acceptance criteria for local aircraft impact effects is that the SC wall is not perforated. This criteria can be satisfied in the limiting case by demonstrating that the tensile strains in the [ ]^{a,c}

[ ] [a,c] does not exceed the limiting values recommended for ferritic steel plates and shells in NEI 07-13 or a conservative value, whether or not there is tearing of the front steel-plate and/or fracture of the tie rod following concrete cracking under impact loading. Therefore, while [ ] [a,c] meant failure in the static load test (red curve) intended to determine out-of-plane shear strength, it does not necessarily mean failure under beyond-design-basis transient aircraft impact. Insights gained from the briefing received by the DPO Appeal working group on the confirmatory global aircraft impact analysis of the SC wall by NRC/RES using a symmetry model of the entire shield building indicate that the progression of impact does result in large deformations from concrete cracking, yielding of the faceplates, yielding and fracture of tie rods around the impact area, and ductile flexural yielding and bulging of the faceplates from dishing of the SC wall, but the strains in the faceplates remain within the failure limits recommended in NEI 07-13. The analysis also indicated rebound of the SC wall after reaching peak deformation as the impact load reduces. The analyses by the DPO Panel consultant (Reference 11), focused on local impact effects, also indicated large deformations without rupture of the steel faceplates. These confirmatory analyses indicated no perforation failure of the SC wall by punching-shear effects from local or global aircraft impact loading, as was concluded by the staff in its inspection (References 9, 27) of WEC's AIA of record.

As indicated above, the use of conventional conservative ACI code-based design equations for two-way shear of reinforced concrete to evaluate punching-shear effects is not realistic for analysis of beyond-design-basis dynamic (transient) aircraft impact loading, and is not recommended in the guidance in Section 2.1.2 of NEI 07-13 even for reinforced concrete structures. The code-based strength equation is overly conservative because it does not account for the transient nature of the aircraft impact loading, the dynamic characteristics of the structure, and strain rate effects involved under impact loading. The required reinforced concrete wall thickness to prevent perforation under local impact loading effects (e.g., punching shear) of an aircraft impact is typically determined in AIAs using the recommended empirical local loading formulas in Section 2.1.2 of NEI 07-13. It should be noted that these formulas are not expressed as a function of the square power of thickness. These formulas are based on experimental data from missile impact tests on flat reinforced concrete panels; not on SC walls. Therefore, consistent with the guidance in RG 1.217, WEC used bench-marked [ ] [a,c] (References 9, 21, and Enclosure 4 of DPO Panel Supplementary Report) capable of capturing different potential failure mechanisms and interactions, where the shield building is modeled in its structural context as a cylindrical structure, to evaluate the local and global effects of aircraft impact loading.

The DPO Appeal working group found it noteworthy that published literature of comparative experimental and analytical studies have confirmed that SC panels have better impact resistant performance than conventional reinforced concrete panels (References 15, 16, 17, 18), enabling the thickness of protection panels to be reduced by approximately 30 percent in comparison with a reinforced concrete panel for similar or higher protective capability (References 15 and 16). Analytical studies by Johnson et al (Reference 17) on load-deformation behavior of impulsively loaded flat SC panels demonstrate that a typical structural response sequence of an SC panel to failure under missile impact is defined by localized failure of the concrete, yielding of the bottom plate, tie bar rupture, and load-displacement strain hardening until the bottom steel plate ruptures. This study also concluded that, while the ductility for SC walls may be less than that of

RC walls, the much larger reinforcement percentage for SC walls affords them superior impact resistance (References 17, 18). An AIA, described in Reference 19, of the CAP1400 shield building, of similar SC construction with a slightly thicker 1.1 m (3.6 ft) cylindrical SC wall of approximately 45.6 m (150 ft) internal diameter and 18.5 inch tie rod spacing (Reference 20) compared to the [ ] [a,c] internal diameter and [ ] [a,c] tie rod spacing for the AP1000 SC wall, concluded that the integrity of the SC wall is maintained after impact with no perforation. This analysis indicates that only a small part of the outer steel plate is damaged and only a small part of the inner steel plate reaches plastic phase with a peak deformation of 130 mm (5.12 inches). The deformation of the inner side is almost the same as the outside and the structure rebounds after peak deformation when the impact load reduces. Although the design and impact parameters in this paper may not be exactly the same as for the AP1000 AIA, the structural characteristics and response reported in this analysis is the best comparable in the open literature to the AP1000 shield building and provides supporting evidence of the performance of cylindrical SC construction under aircraft impact loading. The studies mentioned above further support the conclusions from the aircraft impact analyses performed by the applicant (WEC), the DPO panel consultant, and RES using methodologies consistent with the Force Time-History and/or Missile-Target Interaction analysis methods described in Section 2.2 of NEI 07-13.

The statement of considerations for the 10 CFR 50.150 “Aircraft Impact Assessment” rule (Reference 12) specifies that “realistic analyses” be used. It further states that the NRC may not require, and an interested person in a contention hearing or in a design certification rulemaking comment may not argue, that the designer use a conservative, as opposed to a realistic analysis, or vice versa as long as the designer’s analyses are within the bounds of known data, known physical phenomena, and use professionally-accepted approaches.

With regard to the submitter’s assertion that the DPO Panel consultant’s confirmatory AIA analyses results from the computer codes indicated a bending failure mode, suggesting that the shear failure mode (the red curve) had not been, or had been incorrectly, coded into the computer analytical models, it should be noted that the [ ] [a,c] of the shield building, used by WEC, the DPO Panel consultant, and NRC/RES in their AIAs, [ ] [a,c] (References 9, 11, and interview with NRC/RES staff) and capable of capturing different structural failure limit states. Further, the [ ] [a,c] (Reference 9, and Section 4.2.3.5.3 of DPO Panel Report (Reference 3)). Further, a similar modeling approach was also used in the impact analyses of SC panels or structures documented in References 16 through 19 and Reference 23. Thus, the explicit modeling approach used is professionally-accepted widely, consistent with the guidance in RG 1.217 and NEI 07-13, and was used for global structural aircraft impact assessment of all US design-certification applications reviewed by the NRC thus far, and considered appropriate for realistic best-estimate impact analyses for both local and global effects. Additionally, while the [ ] [a,c] (discussed in the next paragraph) or defining a failure strain limit, in explicit modeling it would not be appropriate to use a single “red curve” from the monotonic shear test of an SC beam module, which does not represent a constitutive relationship nor punching-shear behavior, because the constitutive relationships and strain-based failure criteria are different for the subcomponents of the SC module, for beyond-design-basis aircraft impact analyses involving large deformations. Thus, the analysis results of the DPO panel consultant is considered more representative of the realistic behavior of the shield building SC wall and consistent with the intent of the rule for beyond-design-basis structural assessment under aircraft impact loading.

The AIA of record for the certified AP1000 shield building is that performed by WEC using the LS-DYNA computer code. Section 4.2.3.5.2 of the DPO Panel Report, Enclosure 4 of the DPO Panel Supplemental Report, the NRC Inspection Report No. 05200006/2010-203 for the AP1000 AIA, dated October 28, 2010 (ML102980583), and the ACRS letter report dated January 19, 2011 (ML110210462) document that these analyses were performed using an adequately benchmarked computer code. These documents indicate that the WEC computer code was benchmarked to experimental impact data for SC panels in the paper by Mizuno et al, and the [ ] [a,c] The discussion in Enclosure 4 of the DPO Panel Supplemental Report further indicates that the staff’s additional AIA inspections verified that the steel failure strain limit imposed in the [ ] [a,c] As noted in Enclosure 4 of the DPO Panel supplemental report, the two AIAs performed independently by other experts (i.e., NRC/RES staff and DPO Panel consultant) provides additional assurance as to the ability of the WEC model to produce reasonable results. It should be noted that the AIAs performed by NRC/RES and the DPO Panel Consultant were confirmatory in nature that came to similar conclusions to the WEC AIA.

It is also well understood in structural mechanics that because curvature of a structure provides additional resistance to transverse deformation under load, shell structures resist a large portion of applied lateral load (including impact loading, pressure etc.) by membrane (axial) compression or tension, thereby reducing bending moment and shear forces caused by external load. Evidence of the effect of SC wall curvature and stiffening effect produced by arch action of the cylindrical wall was seen in the early part the impact deformation history in the DPO Panel consultant’s aircraft impact analyses (Reference 11). The solid steel faceplates of the SC wall also afford them additional impact resistance. The DPO panel consultant’s analyses in Reference 11, focused on local aircraft impact effects, was performed using the missile-target interaction analysis method that models the complex interaction of the aircraft with the impacted structure. Since the analysis was focused on vulnerability of the shield building to local punching-shear failure mode, a 180-degree cylindrical wall span was modelled. Fixed boundary is assumed at the base of the cylindrical wall, which is the appropriate condition at the foundation level. Roller supports which allow free radial and axial (vertical) displacements were assumed in the r-z symmetry plane, which is conservative because it maximizes the punching-shear effects. Two boundary condition cases were analyzed for the top boundary of the wall at the spring-line where it joins the conical roof: a totally free boundary, and a totally fixed boundary. The first case allows more bending deformation under impact and maximizes the structural deformation to punching shear loading because part of the energy is absorbed through bending-induced damage in the wall. The top constraints in the second case focus the deformations locally and maximizes the potential for punching shear damage. As indicated by the submitter, the actual boundary condition at the top may be something in between the two cases. However, the boundary conditions used in the analyses in the symmetry plane and at the bottom are appropriate with the two analyses cases for the top boundary providing bounding cases of actual conditions at the top.

As discussed previously, it is noteworthy that the AIA analysis published in the open literature that can be considered most applicable and comparable to that of the [ ] a,c AP1000 shield building [ ] a,c is that of the 3.6 ft thick CAP1400 cylindrical steel-plate concrete composite (SC) shield building (157.2 ft outer diameter and 18.5 inch tie rod spacing) summarized in References 19 and 20. The aircraft considered in the analysis has a weight of 204 tonnes (metric) and a velocity of 156 m/s. This AIA analysis concluded that the structural integrity of the CAP1400 shield building remains intact after impact with a maximum deformation of 130 mm (5.1 in), and provides adequate protection of public health and safety under the postulated aircraft impact. The analysis modeling approach used in the AIAs of the AP1000 shield building is similar to that presented in this paper, and the structural response behavior under aircraft impact documented therein is also qualitatively similar to that observed for the AP1000 SB with larger deformation, providing supplementary supporting evidence that cylindrical SC construction provides adequate performance under impact loading.

The AIA analyses presented in the publication by Katayama, M., Itoh, M., and Rainsberger, R., “Numerical Simulation of Jumbo Jet Impacting on Thick Concrete Walls – Effects of Reinforcement and Wall Thickness (Reference 28),” also using non-linear dynamic finite element explicit modeling, is the impact of a Boeing 747 jet liner with a mass of 340 tonnes and velocity of 300 km/hr [83 m/s] striking a reinforced concrete (RC) flat panel wall150 m wide x 60 m height with thicknesses of 1m, 2m, and 3m. The bottom of the wall is rigidly fixed, while no boundary condition is applied to the other five surfaces (i.e., the target is a cantilever wall). The dynamic response of a structure under transient aircraft impact loading depends on the dynamic characteristics of the structure including the type of construction, mass and material properties, geometric and dimensional parameters, boundary conditions, and impact loading parameters. The construction and the dynamic characteristics of the CAP1400 (and AP1000) shield building SC wall as well as the aircraft impact parameters are different from that of the RC wall panel analyzed in Reference 28; however, it must be highlighted that the impact energy (which is a function of mass and square of the velocity) for the aircraft impact on a cylindrical SC structure in the Reference 19 paper (based on weight 204 t, velocity 156 m/s) is significantly higher (more than 2 times) than that considered in the analyses of the flat RC panel in Reference 28 (based on weight 340 t, velocity 83.3 m/s). Recognizing the differences in construction, comparison of the results of the two analyses provides supporting evidence that the analyzed 1.1 m (3.6 ft) thick cylindrical SC structure in Reference 19 is capable of higher performance under aircraft impact loading than the 2m (6.56 ft) thick reinforced concrete flat panel analyzed in Reference 28. Also, comparing the results from the Reference 19 paper of the 3.6-ft thick cylindrical SC wall to that in Mullapudi’s paper (Reference 23), which indicated perforation of the analyzed 4 ft thick SC flat panel with 1-inch thick faceplates and 0.75-inch tie rods at 17 inch spacing, provides additional supporting evidence of the role of curvature in impact resistance of cylindrical SC structures compared to flat SC panels.

Considering the above, the DPO Appeal working group concludes that the findings in the Office Director's Decision, and responses in the DPO Panel Report and Supplementary Report to the submitter's assertion regarding the punching-shear failure of the SC wall under aircraft impact loading, and their conclusion that the AP1000 shield building will not be perforated by an aircraft impact, is reasonable.

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