Office for Nuclear Regulation

New Reactors Division – Generic Design Assessment

Step 4 Assessment of Civil Engineering for the UK HPR1000 Reactor

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EXECUTIVE SUMMARY

This report presents the findings of my assessment of the civil engineering aspects of the UK HPR1000 reactor design undertaken as part of the Office for Nuclear Regulation's Generic Design Assessment. My assessment was carried out using the Pre-Construction Safety Report and supporting documentation submitted by the Requesting Party.

The objective of my assessment was to make a judgement, from a civil engineering perspective, on whether the generic UK HPR1000 reactor design could be built and operated in Great Britain, in a way that is acceptably safe and secure (subject to site specific assessment and licensing), as an input into ONR's overall decision on whether to grant a Design Acceptance Confirmation.

The scope of the civil engineering generic design assessment was to review the safety aspects of the generic UK HPR1000 design by examining the claims, arguments and supporting evidence in the safety case. My Step 4 generic design assessment built upon the work undertaken in Steps 2 and 3 and enabled a judgement to be made on the adequacy of the civil engineering information contained within the Pre-Construction Safety Report and supporting documentation.

My assessment focussed on the following aspects of the generic UK HPR1000 safety case:

- Output from previous generic design assessment steps
- Civil engineering safety case
- Design principles and methods for reinforced concrete primary structures
- Application of design principles and methods to:
  - Sample 1 – Seismic Category 1 structure on common raft: Fuel building
  - Sample 2 – Internal containment
  - Sample 3 – Common raft foundation
  - Sample 4 – Seismic Category 1 structures on individual rafts: BNX and diesel generator buildings BDB/BDV
  - Sample 5 – Seismic Category 2 structure: BEX
  - Sample 6 – Malicious aircraft impact protection
- Further safety case considerations

The conclusions from my assessment are:

- Residual matters from the Civil Engineering Step 3 assessment have been adequately resolved within this assessment. The Civil Engineering Regulatory Observation raised in Step 2 has been adequately resolved and closed.
- Regarding the civil engineering safety case, the overall structure, scope and limitations are appropriate for generic design assessment, and the cross-cutting inputs are predominantly coherent. The traceability and clarity of the safety functions and the requesting party's use of Safety Functional Requirement schedules are adequate. The requesting party has developed the civil engineering safety case to a proportionate level that meets the purpose of generic design assessment. This provides an adequate reference point from which to develop it more fully in the site-specific phase.
- The civil engineering design principles and methods articulated by the requesting party are appropriate for the purposes of generic design assessment and are adequately aligned with Relevant Good Practice and the intent of the Safety Assessment Principles. These methodologies provide a robust baseline ready for future augmentation to include further detail and site-specific aspects.

- From my assessment of the application of the design principles and methodologies to the six sample areas, I have confirmed that the requesting party has presented an adequate demonstration of the application of its design principles and methodologies.
- Those aspects of novelty, radiation protection, defence in depth, constructability, examination, inspection, maintenance, testing and decommissioning have been adequately considered.
- In summary, for GDA the requesting party has adequately demonstrated the application of Relevant Good Practice and shown via the civil engineering safety case that risks are reduced as low as reasonably practicable.

These conclusions are based upon the following factors:

- A detailed and in-depth technical assessment, on a sampling basis, of the full scope of safety submissions at all levels of the hierarchy of the generic safety case documentation.
- Independent information, reviews and analysis of key aspects of the generic safety case undertaken by Technical Support Contractors.
- Detailed technical interactions with the requesting party, alongside the assessment of the responses to the substantial number of Regulatory Queries and the Regulatory Observations raised during the generic design assessment.

A number of matters remain, which I judge do not undermine the generic UK HPR1000 design and safety submissions but are appropriate for the licensee to consider and take forward in the detailed design and site-specific phases. These are primarily concerned with the provision of site-specific safety case evidence which will become available as the project progresses through the detailed design, construction and commissioning stages. These matters have been captured in 22 Assessment Findings.

Overall, based on my assessment undertaken in accordance with the Office for Nuclear Regulation procedures, the claims, arguments and evidence laid down within the Pre-Construction Safety Report and supporting documentation submitted as part of the generic design assessment process present an adequate safety case for the generic UK HPR1000 design. I recommend that, from a civil engineering perspective, a Design Acceptance Confirmation may be granted.

LIST OF ABBREVIATIONS

ACI	American Concrete Institute
AIA	Aircraft Impact Assessment
ALARP	As Low as Reasonably Practicable
AFCE	French Association for Design, Construction and Surveillance Rules of Nuclear Power Plants Components
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
BDA	Emergency Diesel Generator Building A
BDB	Emergency Diesel Generator Building B
BDC	Emergency Diesel Generator Building C
BDU	Station Black Out Diesel Generator Building U
BDV	Station Black Out Diesel Generator Building V
BEJ	Extra Cooling System and Fire-fighting System Building
BEX	Equipment Access Building
BFX	Fuel Building
BGA	Essential Service Water Supply Gallery A
BGB	Essential Service Water Supply Gallery B
BGC	Essential Service Water Supply Gallery C
BGH	Diesel Buildings Integrated Gallery H
BGI	Diesel Buildings Integrated Gallery I
BGJ	Diesel Buildings Integrated Gallery J
BGL	Essential Service Water Drain Gallery L
BGM	Essential Service Water Drain Gallery M
BGN	Essential Service Water Drain Gallery N
BGT	Liquid Waste Transfer Gallery
BMX	Turbine Generator Building
BNX	Nuclear Auxiliary Building
BPA	Essential Service Water Pumping Stations A
BPB	Essential Service Water Pumping Stations B
BPX	Personnel Access Building
BoD	Basis of Design
BoSC	Basis of Safety Case
BRB	Bradwell B Site
BRX	Reactor Building
BSA	Safeguard Building A
BSB	Safeguard Building B
BSC	Safeguard Building C

BSI	British Standards Institution
BSR	Barrier Substantiation Report
BWX	Radioactive Waste Treatment Building
CDFM	Conservative Deterministic Failure Margin
CDM 2015	Construction (Design and Management) Regulations 2015
CGN	China General Nuclear Power Corporation Ltd
CM9	ONR tool for electronic file storage (reference numbers for location)
DAC	Design Acceptance Confirmation
DB	Design Basis
DBA	Design Basis Accident
DBE	Design Basis Event
DMGL	Delivery Management Group Lead
DR	Design Reference
DRR	Design Risk Register
DSR	Design Substantiation Report
EC	External Containment
EIMT	Examination, Inspection Maintenance and Testing
FE	Finite Element
FRS	Floor Response Spectra
GDA	Generic Design Assessment
HEPF	High energy pipe failure
HOW2	(ONR) Business Management System
HCLPF	High Consequence of Low Probability of Failure
HVAC	Heating, Ventilation and Air Conditioning
IAEA	International Atomic Energy Agency
IC	Internal Containment
ISO	International Organisation for Standardisation
LLSF	Low Level Safety Function
LOCA	Loss of Coolant Accident
NEI	Nuclear Energy Institute
NPP	Nuclear Power Plant
NUREG	United States Nuclear Regulatory Commission Regulation Regulatory Report
ONR	Office for Nuclear Regulation
PCSR	Pre-construction Safety Report
PSA	Probabilistic Safety Analysis
RGP	Relevant Good Practice
RI	Regulatory Issue
RO	Regulatory Observation
RP	Requesting Party

RQ	Regulatory Query
SAP(s)	Safety Assessment Principle(s)
SDMS	Structural Analysis and Design Method Statement
SFP	Spent Fuel Pool
SFR	Safety Functional Requirement
SSCs	Structures, Systems and Components
SSE1/SSE2	Safety categorisation for structures with or without seismic classification
SSI	Soil-Structure Interaction
SSSI	Structure-Soil-Structure Interaction
TAG	Technical Assessment Guide(s)
TSC	Technical Support Contractor
UK HPR1000	UK Version of the Hua-long Pressurised Reactor
USNRC	United States Nuclear Regulatory Commission
WENRA	Western European Nuclear Regulators' Association

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INTRODUCTION

1.1 Background

1. This report presents my assessment conducted as part of the Office for Nuclear Regulation (ONR) Generic Design Assessment (GDA) for the generic UK HPR1000 design within the topic of civil engineering.

2. The UK HPR1000 is a pressurised water reactor (PWR) design proposed for deployment in the UK. General Nuclear System Ltd is a UK-registered company that was established to implement the GDA on the UK HPR1000 design on behalf of three joint requesting parties (RP), i.e., China General Nuclear Power Corporation (CGN), EDF SA and General Nuclear International Ltd.

3. GDA is a process undertaken jointly by the ONR and the Environment Agency. Information on the GDA process is provided in a series of documents published on the joint regulators' website (www.onr.org.uk/new-reactors/index.htm). The outcome from the GDA process sought by the RP is a Design Acceptance Confirmation (DAC) for ONR and a Statement of Design Acceptability from the Environment Agency.

4. The GDA for the generic UK HPR1000 design followed a step-wise approach in a claims-argument-evidence hierarchy which commenced in 2017. Major technical interactions started in Step 2 which focussed on an examination of the main claims made by the RP for the UK HPR1000. In Step 3, the arguments which underpin those claims were examined. The Step 2 reports for individual technical areas, and the summary reports for Steps 2 and 3 are published on the joint regulators' website. The objective of Step 4 was to complete an in-depth assessment of the evidence presented by the RP to support and form the basis of the safety and security cases.

5. The full range of items that form part of my assessment is provided in ONR's GDA Guidance to Requesting Parties (Ref. 1). These include:

- Consideration of issues identified during the earlier Step 2 and 3 assessments.
- Judging the design against the Safety Assessment Principles (SAPs) (Ref. 2) and whether the proposed design demonstrates that risks are As Low As Reasonably Practicable (ALARP).
- Reviewing details of the RP's design controls and quality control arrangements to secure compliance with the design intent.
- Establishing whether the system performance, safety classification, and reliability requirements are substantiated by a more detailed engineering design.
- Assessing arrangements for ensuring and assuring that safety claims and assumptions will be realised in the final as-built design.
- Resolution of identified nuclear safety and security issues, or identifying paths for resolution.

6. The purpose of this report is therefore to summarise my assessment in the civil engineering topic which provides an input to the ONR decision on whether to grant a DAC, or otherwise. This assessment was focused on the submissions made by the RP throughout GDA, including those provided in response to the Regulatory Queries (RQs), Regulatory Observations (ROs) I raised. I did not raise any Regulatory Issues. Any RIs and ROs issued to the RP are published on the GDA's joint regulators' website, together with the corresponding resolution plans.

1.2 Scope of this Report

7. This report presents the findings of my assessment of the civil engineering of the generic UK HPR1000 design undertaken as part of GDA. I carried out my assessment using the Pre-construction Safety Report (PCSR) Ref. 3 and supporting documentation submitted by the RP. My assessment was focussed on considering whether the generic safety case provides an adequate justification for the generic UK HPR1000 design, in line with the objectives for GDA.

1.3 Methodology

8. The methodology for my assessment follows ONR's guidance on the mechanics of assessment, NS-TAST-GD-096 (Ref 4).

9. My assessment was undertaken in accordance with the requirements of ONR's How2 Business Management System (BMS). ONR's SAPs (Ref. 2) together with supporting Technical Assessment Guides (TAG) (Ref. 4), were used as the basis for my assessment. Further details are provided in Section 2 below. The outputs from my assessment are consistent with ONR's GDA Guidance to RPs (Ref. 1).

2 ASSESSMENT STRATEGY

10. The strategy for my assessment of the civil engineering aspects of the UK HPR1000 design and safety case is set out in this section. This identifies the scope of the assessment and the standards and criteria that have been applied.

2.1 Assessment Scope

11. A detailed description of my approach to this assessment can be found in the civil engineering Step 4 assessment plan (Ref. 5), which underwent minor optimisation to account for input from other disciplines and matters arising. The final assessment scope definition is provided herein, with the corresponding RP submissions described in Section 3.

12. I considered all of the main RP submissions within the remit of my assessment scope, to various degrees of breadth and depth. I chose to concentrate my assessment on those aspects that I judged to have the greatest safety significance, or where the hazards appeared least well controlled. My assessment was also influenced by the claims made by the RP, my previous experience of similar systems for reactors and other nuclear facilities, and any identified gaps in the original submissions made by the RP. A particular focus of my assessment has been the RQs (Ref. 6), ROs (Ref. 7) I raised as a result of my on-going assessment, and the resolution thereof.

2.2 Sampling Strategy

13. In line with ONR's technical guidance, see Section 2.4.2 for specific documents, the scope of my assessment was developed on a sampling basis. My sampling strategy is based upon:

- Review of the outputs from previous UK HPR1000 GDA Step 2 (Ref. 8) and Step 3 (Ref. 9). This included RQ's, RO's and 'Open Points' raised as a result of my on-going assessment, and the resolution thereof.
- Review of previous GDA Step 4 scopes and ONR guidance for GDA Step 4 to ensure a consistent and proportionate assessment coverage1.
- Identification of sample structures for in-depth assessment of the application of the GDA Step 3 methodologies based on:
  - their contribution to nuclear safety;
  - the technical complexity / novelty of their structural arrangements; and
  - the performance requirements under design basis, beyond design basis and severe accident states.
- Consultation with other ONR disciplines to establish common cross-cutting sample areas.
- Review and refinement by project stakeholders to incorporate previous GDA learning and experience[2].

14. The broad areas of the scope are described in more detail below.

Outputs from Previous GDA Steps

15. Following on from the previous GDA Steps, the more significant risk area for civil engineering regarding the geotechnical design parameters and associated analysis methodology were formally recorded in an RO, RO-UKHPR1000-0009 (Ref. 7). RO-UKHPR1000-0009 was the only civil engineering RO. This and RO's from other disciplines that had civil engineering input are summarised in Table 5 below.

[1] *Previous projects include the GDA's for EPR, AP1000 and ABWR.*
[2] *This included the supporting ONR TSC experts as well as ONR inspectors.*

16. At the end of Step 3, Ref. 9 denoted other outstanding matters as *'Areas for Improvement'* and/or *'Open Points'*. All of the *'Areas for Improvement'* and some of the *'Open Points'* were captured by RQ's as shown in Annex 3 below, and Tables 1 & 2 of Ref. 9. Those *'Open Points'* not captured by RQ's (see Table 2 of Ref. 9) were incorporated into the ONR Step 4 assessment plan for civil engineering (Ref. 5). Table 5 below articulates the relevant sections of this assessment that cover these areas.

Civil Engineering Safety Case

17. The assessment of Step 4 continued the examination of the civil engineering safety case. This included the assessment of the following areas.

- Assessment of the overarching safety case framework and document structure for civil engineering.
- The development, identification and traceability of the Structures, Systems and Components (SSC), Safety Functional Requirements (SFR), and engineering requirements for structures, including requirements arising from other disciplines.
- Cliff edge and beyond design basis conditions.
- The mechanism for communicating the safety case requirements through the design process and reporting.
- The substantiation or fulfilment of the SFRs and its communication.
- The completeness and clarity of the forward actions.
- The overall quality assurance applied to all of the above.

18. The assessment of the above considered the extant civil engineering documentation and focussed in specifically on the sample structures to test the application of the safety case framework.

Design Principles and Methods for reinforced concrete primary structures

19. The overarching design principles and methodologies were reviewed in Step 3 (Ref. 9). However, some of the very detailed aspects were not fully explicated by the RP and required further examination in Step 4. Examples of these areas are as follows:

- The integration and application of local analysis models within the methodologies.
- The complete process of verification and validation to complement the analysis and design methodologies.
- Further details on the methods used within the ReinCal software for strength design.

Application of design principles and methods

20. I sampled three different structures to assess the demonstration of the above safety case and design principles and methods. The structures I sampled were the Fuel Building (BFX), the Internal Containment, and the Common Raft Foundation. A description of these structures is provided in Annex 5 of this report.

21. However, these three structures are all founded on the common raft, are deemed to have a relatively shallow embedment, and are of Class 1 or SSE1 classification (see Figure 5, para. 76). Therefore, aspects of the principles and methods that apply to other structures with different characteristics would not be demonstrated. Therefore, these three samples were augmented with samples of other structures to ensure adequate assessment coverage for GDA. These were the BNX, BDB/BDV and BEX, and the reasons for their inclusion are explicated below.

22. For malicious aircraft impact assessment, the complete safety case was assessed for all structures providing impact protection.

23. The justification for each of the sample areas is provided below.

Sample 1 – BFX (SSE1 structure on common raft)

24. This structure is considered representative of a Class 1 structure located on the common raft. It comprises a cellular reinforced concrete structural form for which the analysis, design methodology, and challenges are expected to be very similar to other structures on the common raft. Therefore, the application of the design principles and methodologies to the BFX is expected to provide sufficient confidence for other Class 1 structures, such as the safeguard buildings and BRX.

25. The BFX contains the Spent Fuel Pond (SFP) which provides containment to a significant radiological inventory in the form of spent fuel assemblies. The SFP structure, including the stainless-steel liner, needs to be substantiated to meet the SFRs for design basis and beyond design basis conditions, maintaining the containment function for all loading conditions.

Sample 2 - Internal Containment

26. The internal containment is a structural barrier to prevent of release of radiological material. It is the only structure to comprise a post-tensioned concrete construction, with in-situ grouted tendons and an integrated steel liner. Furthermore, it is a complex structural system, and the analysis models are sophisticated to demonstrate the design can be substantiated against SFRs for design basis, beyond design basis and severe accident conditions. This requires the development, verification, and validation of complex global and local finite element models that can predict both the elastic and inelastic behaviour, including the likely failure modes. Therefore, the internal containment modelling philosophy and its validation are complex. In conjunction with the deterministic analysis and application design codes, these finite element models are also applied within a probabilistic framework to develop fragility curves to represent the internal containment performance. The results are significant contributors to the both the PSA and severe accident disciplines. Furthermore, the internal containment EIMT requirements for strain gauges are directly underpinned by the tendon failure analysis.

Sample 3 - Common Raft Foundation

27. This structure forms a barrier to the potential release of radiological material to the ground and is fundamental to the integrity of the structures and systems it supports. The analysis and modelling require the geotechnical properties of the generic site envelope[3] to be characterised both statically and dynamically. The methodologies for the raft with respect to the overall stability analysis and computation of settlements and tilts, are unique, and require static and dynamic soil-structure-interaction (SSI) to be considered. The geometry directly influences the settlements and inclinations that are a critical input for the design of SSCs.

Sample 4 – BNX and BDB/BDV facilities (SSE1 structures on individual rafts)

28. Some of these buildings are immediately adjacent to the common raft, and therefore susceptible to the effects of structure-soil-structure interaction (SSSI). The BNX is a non-symmetrical structure situated immediately adjacent to the common raft and was

[3] Although the GDA process does not consider site-specific geotechnical parameters, the results using the generic site envelope enable a judgment to be made regarding the foundation concept, and whether it is suitable to be progressed and optimised further in the site-specific phase.

selected to be sampled to assess this aspect of the RP's design and analysis methodology.

29. The BDB/BDV and BDU/BDC/BDA are spatially separated from, and less susceptible to SSSI effects from, the common raft foundation. Regarding embedment, these buildings have higher depth-to-equivalent-radius ratios[4], compared to the other Class 1 structures supported upon the common raft. The joined BDB/BDV building was sampled to look specifically at the application of the RPs seismic analysis methodology when considering embedment effects. Further, the Diesel Generator Buildings are susceptible to hydrocarbon fire and the design methodology to combat this has been sampled within the context of BDB/BDV.

Sample 5 – BEX (SSE2 structure)

30. The two non-classified structures within the GDA scope (BEX and BPX) are smaller in size and have simpler configurations than other buildings. The focus of this sample was to confirm the differences in analysis and design methodologies to those used for the Class 1 structures. Specifically, a focus of the assessment was to demonstrate the design of these non-classified structures did not adversely affect the SFRs of safety critical SSCs. The BEX was chosen for this sample.

Sample 6 - Malicious Aircraft Impact Protection

31. This hazard poses a risk to safe operation and containment of radiological material. Furthermore, the design analysis can have significant implications to the layout and geometry of the civil structures and the SSC's housed within. The load derivation, analysis and design employ specific methodologies that are specialist in nature. Therefore, all structures providing aircraft impact protection have been sampled.

<u>Further safety case considerations</u>

32. Further to the above, other areas that have been assessed, with the purpose of de-risking the future site-specific stages, are as follows:

- The evaluation of the below ground water-proofing system, including the approach, material, sliding performance, ageing effect and durability aspects have been sampled specifically for the common raft and other deeply embedded structures more generally.
- The constructability of civil engineering structures, along with the general application of Construction Design Management (CDM) principles.
- The in-service examination, inspection, maintenance, and testing (EIMT) for civil engineering have been assessed, including the Internal Containment inspection and pressure testing requirements.
- The definition and change control process for the GDA design for civil engineering including the use of drawings and the proposed 3D Plant Design Management System (PDMS) model.
- The general proposed approach to future decommissioning.

2.2.1 Engagement Strategy

33. The GDA Step 4 Assessment Plan (Ref. 5) outlines a series of technical workshops and progress meetings that would be held across the assessment period.

[4] This ratio is used to determine whether embedment effects should be considered in the analysis, see ASCE 4-16 for further information.

34. For the first half of Step 4, the engagements were focused on ONR gaining knowledge and insight from the RP. Workshop style engagements were used to accomplish this objective, each of which applied the following process:

- Review the relevant RP submissions, including any cross-cutting topic areas.
- Carry out horizon scanning of relevant good practice and develop expectations.
- Develop a set of subject specific themes to guide the RP's preparation of the technical presentations.
- Provision of the appropriate team of experts to support ONR during the workshop.
- Make a record of the discussions in Contact Records.
- Record and issue all residual technical matters in RQ's to facilitate progression to closure.

35. A summary of the technical workshops held during GDA Step 4 is provided in Table 1 below.

Table 1: Summary of technical workshops held during Step 4

Workshop No.	Topic	Date
#01	Safety Case Management, SFR Schedules, Seismic Analysis, Common Raft, BFX,	30 March-3 April 2020
#02	Aircraft Impact Safety Case, Extent of Aircraft Impact protection	28-29 April 2020
#03	Safety Case and golden thread, Inner Containment finite element modelling, prestressing, modelling and strength design, liner design, static SSI for RO-UKHPR1000-0009, aircraft impact	1-5 and 8 June 2020
#04	Local modelling, Design Process walkthrough, internal hazards, spent fuel pond design	1-8 July 2020
#05	Safety Case and information management, barriers and internal hazard interface, internal containment liner, common raft and geotechnics, design of structures off common raft	27-31 July 2020
#06	Aircraft impact analysis, fire spread and shielding	3 September 2020
#07	Structural and seismic verification and validation, sensitivity studies, internal containment sensitivity and verification and validation	14, 16, 18 and 21 September 2020
#08	Mid Step review feedback, internal containment gusset design work, analysis methodology for discontinuous regions, tendon failure analysis, thermal reduction factors, BFX modifications, internal hazards decoupling approach	4, 6 November 2020
#09	Shielding, fire spread analysis	3, 5 November 2020
#10	Beyond Design Basis and cliff edge effects of extreme environmental hazards, Internal containment ultimate capacity evaluation	19, 21, 22 January 2021
#11	Internal containment fragility analysis	8 February 2021
#12	ALARP and CDM Workshop	5 March 2021

36. The ONR Civil Engineering team held a mid-step review of progress in October and November 2020. Following this, the focus shifted from workshops to a series of weekly RQ technical progress meetings to focus on the resolution of RQ's and technical areas of concern. The last of these meetings was held on 31st March 2021. A summary of these meetings is provided in Table 2 below.

Table 2: Summary of RQ technical progress meetings

Meeting No.	Topic	Date
#01	Aircraft Impact mid step review feedback, RQ-UKHPR1000-0852, RQ-UKHPR1000-0889, RQ-UKHPR1000-0939 and RQ-UKHPR1000-0948	20 November 2020
#02	Internal containment gusset design, RQ-UKHPR1000-1298 and RQ-UKHPR1000-0889	27 November 2020
#03	General progress update for RQ-UKHPR1000-1277, RQ-UKHPR1000-0890, RQ-UKHPR1000-1044, RQ-UKHPR1000-1324, RQ-UKHPR1000-1123 and RQ-UKHPR1000-1156	4 December 2020
#04	Internal containment gusset design	11 December 2020
#05	Static SSI, RQ-UKHPR1000-1160, RQ-UKHPR1000-1357, RQ-UKHPR1000-1345, Internal containment RQ-UKHPR1000-1157, RQ-UKHPR1000-1320, and RQ-UKHPR1000-1321	18 December 2020
#06	Internal containment gusset design and thermal reduction factor justification	8 January 2021
#07	Internal containment supplementary calculations, tendon failure analysis	15 January 2021
#08	Internal containment liner, RQ-UKHPR1000-0857, RQ-UKHPR1000-0862, RQ-UKHPR1000-1273, RQ-UKHPR1000-1274 and RQ-UKHPR1000-1272.	29 January 2021
#09	Internal containment gusset design and thermal reduction factor justification, RQ-UKHPR1000-1430, RQ-UKHPR1000-1433, supplementary calculations for the ultimate capacity assessment	5 February 2021
#10	Static SSI settlements and credible solutions, RQ-UKHPR1000-1357, RQ-UKHPR1000-1160.	19 February 2021
#11	Aircraft impact, RQ-UKHPR1000-1329, RQ-UKHPR1000-1330, RQ-UKHPR1000-1379, RQ-UKHPR1000-1540, RQ-UKHPR1000-1333, Internal containment gusset design RQ-UKHPR1000-1430 and RQ-UKHPR1000-1431	26 February 2021
#12	Internal containment ultimate capacity evaluation and fragility derivation, RQ-UKHPR1000-1523, RQ-UKHPR1000-1522, RQ-UKHPR1000-1523, RQ-UKHPR1000-1524 and RQ-UKHPR1000-1486	12 March 2021
#13	Internal containment ABAQUS local model, shear design for gusset region and RQ-UKHPR1000-1433.	19 March 2021
#14	General progress update for RQ-UKHPR1000-1533, RQ-UKHPR1000-1567, RQ-UKHPR1000-0890, RQ-UKHPR1000-0948 and RQ-UKHPR1000-1526.	25 March 2021
#15	General progress update for RQ-UKHPR1000-1522, RQ-UKHPR1000-1523, RQ-UKHPR1000-1485, RQ-UKHPR1000-1486, RQ-UKHPR1000-1488, RQ-UKHPR1000-1433 and RQ-UKHPR1000-1540.	31 March 2021

37. After 1st April 2021, during the ONR assessment period, one further meeting was held to discuss outstanding technical matters and to confirm the RP’s position where uncertainty remained. The engagements and associated documentation are recorded in a tracker (Ref. 10).

2.2.2 Limitations

38. For civil engineering, it is acknowledged that the site-specific structural design will be heavily influenced by the site-specific conditions associated with the *'Target Site'*. Therefore, the GDA design, based on a generic site envelope, is expected to be developed and optimised significantly during the site-specific phase to account for the numerous site and construction specific factors that cannot be constrained for GDA.

39. For this civil engineering assessment, the analysis and substantiations are therefore not taken to full construction-approved level of detail. Rather, the aim is to demonstrate that the level of detail is sufficient to demonstrate that the safety functions can be underpinned. This is to include demonstration that any risk areas can be understood, discussed and associated design modifications or other improvements implemented or committed to. The GDA Step 4 process tests the RP's ability to articulate and apply their suite of Step 3 methodologies, demonstrating the competence and experience of their design team and the robustness of the design.

40. With the above in mind, along with the declared out of scope items, the GDA for civil engineering has limitations. A non-exhaustive list of examples is as follows:

- Use of generic site conditions and design parameters
- The methodologies do not fully detail all aspects of the site-specific work
- Generic foundation concept, which will be influenced by site specific conditions
- Use of generic external hazard inputs, with exclusions such as site flooding and the associated effects: therefore, the hazard screening is generic.
- Limited application of local computational models used in design analysis calculations
- Limited design outputs based on assessment of sampled structural elements
- Construction level information e.g., reinforcement detailing drawings, is very limited and in general not provided

2.3 Out of Scope Items

41. The following items were outside the scope of my assessment:

- Those structures that the RP declared as dependent on site-specific inputs, including:
    - The Essential Service Water Pumping Stations (BPA, BPB), the Circulating Water Pumping Station (BPW), the Essential Service Water Supply Galleries (BGA, BGB, BGC), the Diesel Buildings Integrated Galleries (BGH, BGI, BGJ), the Essential Service Water Drain Galleries (BGL, BGM, BGN), and the Turbine Generator Building (BMX).
- GDA is based on a generic site layout for UK HPR1000 with a single unit reactor design: therefore, multi-unit considerations are not considered.
- The activities subject to detailed design / site specific consideration are out of scope of GDA, a non-exhaustive list of examples is as follows:
    - Construction sequencing and temporary works
    - Concrete construction properties associated with hydration, creep, thermal effects and shrinkage
    - Concrete mix design and specification
    - Mechanical anchorages and embedded components
    - Non-concrete and secondary structures, e.g., the BFX roof-mounted stack, steel walkways and access platforms
    - Reinforcement detailing
    - Temporary works design
    - Other secondary structural components, e.g., doors
    - Contaminated ground and associated durability considerations.

- Embedded services
- External waterproofing and rainwater systems

2.4 Standards and Criteria

42. The relevant ONR guidance adopted within this assessment are principally the Safety Assessment Principals (SAPs) and Technical Assessment Guidance (TAGs). Also adopted are the relevant national and international standards and other relevant good practice (RGP) informed from existing practices adopted on nuclear licensed sites in Great Britain. The key SAPs and any relevant TAGs, national and international standards and guidance are detailed within this section. RGP is cited within Section 4 where applicable.

2.4.1 Safety Assessment Principles

43. The SAPs (Ref. 2) constitute the regulatory principles against which ONR judge the adequacy of safety cases. The SAPs applicable to civil engineering are included within Annex 1 of this report.

44. The Civil Engineering (ECE) suite of SAPs that applied within my assessment were namely:

- Principles ECE.1-3
- Site Investigation ECE.4,5
- Design ECE.6-15
- Construction ECE.16-19, 25
- EIMT ECE.20-24
- Decommissioning ECE.26

45. The other key SAPs applied within my assessment are the suites of SAPs associated with safety case (SC), safety classification and standards (ECS), key engineering principles (EKP), external and internal hazards (EHA), reliability (EDR and ERL), layout (ELO), ageing and degradation (EAD) and assurance of validity of data and models (AV).

2.4.2 Technical Assessment Guides

46. The following Technical Assessment Guides (Ref. 4) were used as part of this assessment:

- NS-TAST-GD-096, *Guidance on Mechanics of Assessment*.
- ONR-TAST-GD-017, *Civil Engineering (with ONR-TAST-GD-076 'Construction Assurance' subsumed as an annex of TAG 017)*
- ONR-TAST-GD-020, *Civil Engineering for Containments for reactor plant*
- ONR-TAST-GD-013, *External Hazards*
- ONR-TAST-GD-014, *Internal Hazards*
- ONR-TAST-GD-094, *Categorisation of safety functions and classification of SSCs*
- NS-TAST-GD-051, *The Purpose, Scope and Content of Nuclear Safety Cases*
- NS-TAST-GD-005, *ONR Guidance on the Demonstration of ALARP*

2.4.3 National and International Standards and Guidance

47. The following standards and guidance were used as part of this assessment:

- International Atomic Energy Agency (IAEA) Specific Safety Requirements, Specific Safety Guides, General Safety Guides, Safety Guides and Safety Report Series
- Western European Nuclear Regulators' Association (WENRA) Safety Reference Levels
- United States Nuclear Regulatory Commission (USNRC) guidance and associated regulation (Nuclear Regulatory Commission Regulation Reports and Guidance, 'NUREG')
- American Society of Civil Engineers (ASCE) codes and standards
- American Society of Mechanical Engineers (ASME) codes and standards
- American Concrete Institute (ACI) codes and standards
- Eurocodes (EN)
- Nuclear Energy Institute (NEI) methodology for performing Aircraft Impact assessments
- French Association for Design, Construction and Surveillance Rules of Nuclear Power Plants Components (AFCEN)

48. The specific codes and standards referenced in this assessment are presented in Annex 2.

2.5 Use of Technical Support Contractors

49. For civil engineering, it is necessary in GDA for ONR to use Technical Support Contractors (TSCs) to provide additional capacity and access independent expert technical advice and experience for specialist subject areas. This enables ONR's inspectors to focus on regulatory decision making.

50. Table 3 below sets out the areas in which I used TSCs to support my assessment.

Table 3: Work packages undertaken by the TSC

Number	Description
1	The overall objective of this work was to provide technical support to ONR, in order to review the RP civil engineering submissions and aircraft impact safety case. The scope of work involved providing an independent technical review of the civil engineering safety case, including the sample topic areas of the inner containment, aircraft impact, common raft foundation, structures on the common raft, structures on adjacent individual raft and non-classified structures. Other areas of focus and topics were cross-cutting into other disciplines, such as internal and external hazards, fault studies and Probabilistic Safety Assessment.

51. Whilst the TSC undertook detailed technical reviews, this was done under my direction and close supervision. The regulatory judgment on the adequacy, or otherwise, of the generic UK HPR1000 safety case in this report has been made exclusively by me, informed where appropriate by the TSC's expert advice.

2.6 Integration with Other Assessment Topics

52. GDA requires the submission of an adequate, coherent and holistic generic safety case. Regulatory assessment cannot be carried out in isolation as there are often issues that span multiple disciplines. I have therefore worked closely with several other ONR inspectors to inform my assessment. The key interactions were:

- External Hazards took the lead in confirming the adequacy of the civil engineering design basis definitions and load functions from external hazards. I took the lead in assessing the link from external hazard safety case claims into the civil engineering safety case and design substantiation. I provided civil engineering input to support the assessment of RO-UKHPR1000-0002 'Demonstration that the UK HPR1000 Design is Suitably Aligned with the Generic Site Envelope', and RO-UKHPR1000-0007 'Aircraft Impact Safety Case of UK HPR1000', (Ref. 7) and external hazards provided input to Action 1 of RO-UKHPR1000-0009 'Geotechnical Design Parameters' (Ref. 7).
- Internal Hazards took the lead in ensuring that civil engineering barriers have been appropriately identified and that the methodology for developing the civil engineering load functions is appropriate. I took the lead in ensuring the link from the internal hazard safety case claims into the civil engineering safety case and design substantiation is explicit and navigable. I provided civil engineering input to support the assessment of RO-UKHPR1000-0054 'Validation of Internal Hazard loadings used for Civil Engineering design of non-barrier elements' (Ref. 7).
- Fault Studies took the lead in confirming that the pressure and temperature load functions used for design of the internal containment are appropriate and to demonstrate the adequacy of the RP's approach to the categorisation of safety functions and classification of SSC's. I took the lead in assessing the completeness of inputs to the SFR schedules.
- Probabilistic Safety Assessment (PSA): I took the lead in assessing the fragility functions used in the modelling of pressure loads on the containment. PSA took the lead on the application of these functions into the PSA.
- Mechanical Engineering: I took the lead in clarifying the boundaries of responsibility between the two respective disciplines for generation of floor response spectra, embedded details such as the Fuel Transfer Tube (FTT) and secondary structural components e.g., polar crane bracket. I liaised with mechanical engineering regarding RO-UKHPR1000-0014 'Spent Fuel Building – Design of Nuclear Lifting Operations to Demonstrate Relevant Risks are Reduced to ALARP' (Ref. 7).
- Structural Integrity: I took the lead in clarifying the boundaries of responsibility between the two respective disciplines for details such as the FTT, BRX internal structures, and also the input required during Step 4 for assessing the internal containment liner and other structures internal to the containment. I provided civil engineering input to support the assessment of RO-UKHPR1000-0008 'Justification of the Structural Integrity Classification of the Main Coolant Loop' and RO-UKHPR1000-0058 'Justification of the Structural Integrity Classification of the UK HPR1000 Main Steam Line and Associated Major Valves in the Safeguards Buildings' (Ref. 7).
- Conventional Health and Safety: I took the lead in ensuring a consistent position regarding constructability and the application of the CDM regulations for civil engineering structures, where conventional health and safety completed a more wide-ranging assessment of CDM across all the GDA scope.
- All other ONR disciplines were engaged to gain input on cross cutting issues such as the adequacy of the safety case, spatial provision, layout and EIMT. Related to these areas, I provided civil engineering input to support the assessment of RO-UKHPR1000-0004 and RO-UKHPR1000-0056 'Fuel Route Safety Case' (Ref. 7).

2.7 Overseas Regulatory Interface

53. ONR has formal information exchange agreements with a number of international nuclear safety regulators and collaborates through the work of the International Atomic Energy Agency (IAEA) and the Organisation for Economic Co-operation and Development Nuclear Energy Agency. This enables us to utilise overseas regulatory

assessments of reactor technologies, where they are relevant to the UK. It also enables the sharing of regulatory assessments, which can expedite assessment and helps promote consistency.

2.7.1 Bilateral Collaboration

54. ONR requested further information informally from the United States Nuclear Regulatory Committee (USNRC) specialists on points of detail regarding the application of US-based relevant good practice (RGP). This included the seismic analysis, and the interpretation of the NEI 07-13_8P guidelines for aircraft impact, and the interpretation of the ACI349-13 code rules for design of 2-D concrete elements. Where relevant this is referred to in Section 4. However, no formal collaboration was necessary.

2.7.2 Multilateral Collaboration

55. No formal collaboration was necessary for my assessment.

3 REQUESTING PARTY'S SAFETY CASE

3.1 Introduction to the Generic UK HPR1000 Design

56. The generic UK HPR1000 design is described in detail in the PCSR (Ref. 3). It is a three-loop PWR designed by CGN using the Chinese Hualong technology. The generic UK HPR1000 design has evolved from reactors which have been constructed and operated in China since the late 1980s, including the M310 design used at Daya Bay and Ling'ao (Units 1 and 2), the CPR1000, the CPR1000^{+} and the more recent ACPR1000. The first two units of CGN's HPR1000, Fangchenggang Nuclear Power Plant Units 3 and 4, are under construction in China and Unit 3 is the reference plant for the generic UK HPR1000 design. The design is claimed to have a lifetime of at least 60 years and has a nominal electric output of 1,180 Megawatts.

57. The reactor core contains zirconium clad uranium dioxide (UO_2) fuel assemblies and reactivity is controlled by a combination of control rods, soluble boron in the coolant and burnable poisons within the fuel. The core is contained within a steel Reactor Pressure Vessel (RPV) which is connected to the key primary circuit components, including the Reactor Coolant Pumps (RCPs), Steam Generators (SGs), pressuriser and associated piping, in the three-loop configuration. The design also includes a number of auxiliary systems that allow normal operation of the plant, as well as active and passive safety systems to provide protection in the case of faults, all contained within a number of dedicated buildings.

58. The reactor building (BRX) houses the reactor and primary circuit and is based on a double-walled containment with a large free volume. Three separate safeguard buildings (BSA, BSB, BSC) surround the reactor building and house key safety systems and the main control room. The fuel building (BFX) is also adjacent to the reactor building and contains the fuel handling and short-term storage facilities. Finally, the nuclear auxiliary building (BNX) contains a number of systems that support operation of the reactor. In combination with the diesel (BDA, BDB, BDC), personnel access (BPX) and equipment access (BEX) buildings, these constitute the nuclear island for the generic UK HPR1000 design.

59. An overview of the nuclear safety related civil engineering structures that are included in the GDA Step 4 scope is provided in Ref. 11. This document provides concept level illustrations and diagrams with some example structural details and supporting narrative. Some of this content is captured within Annex 5 of this report. This annex contains diagrams and descriptions of the generic UK HPR1000 nuclear island structures that have formed the main scope of this GDA Step 4 assessment.

3.2 The Generic UK HPR1000 Civil Engineering Safety Case

60. In this section I provide an overview of the generic UK HPR1000 civil engineering safety case that has been formally submitted by the RP during GDA. Further details and references to the specific technical content within the RP's documentation that pertain to my assessment are provided in Section 4 of this report.

3.2.1 Documentation structure and golden thread

61. The safety case structure and golden thread is described in the 'Production Strategy' (Ref. 12). This describes the tasks and deliverables in GDA that make up the safety case information for GDA Step 4 and presents an overview of the civil engineering safety case document structure hierarchy and the golden thread. Other aspects of the civil engineering safety case are presented, including the objective of the UK HPR1000 GDA, scope, organisational resource, risk management, safety case update strategy and document list.

62. The rationale or flow of the safety case construct and how this links to the various documents is illustrated by F-4-12 of Ref. 12, an extract of this is shown in Figure 1 below. A larger legible version is included in Annex 6.

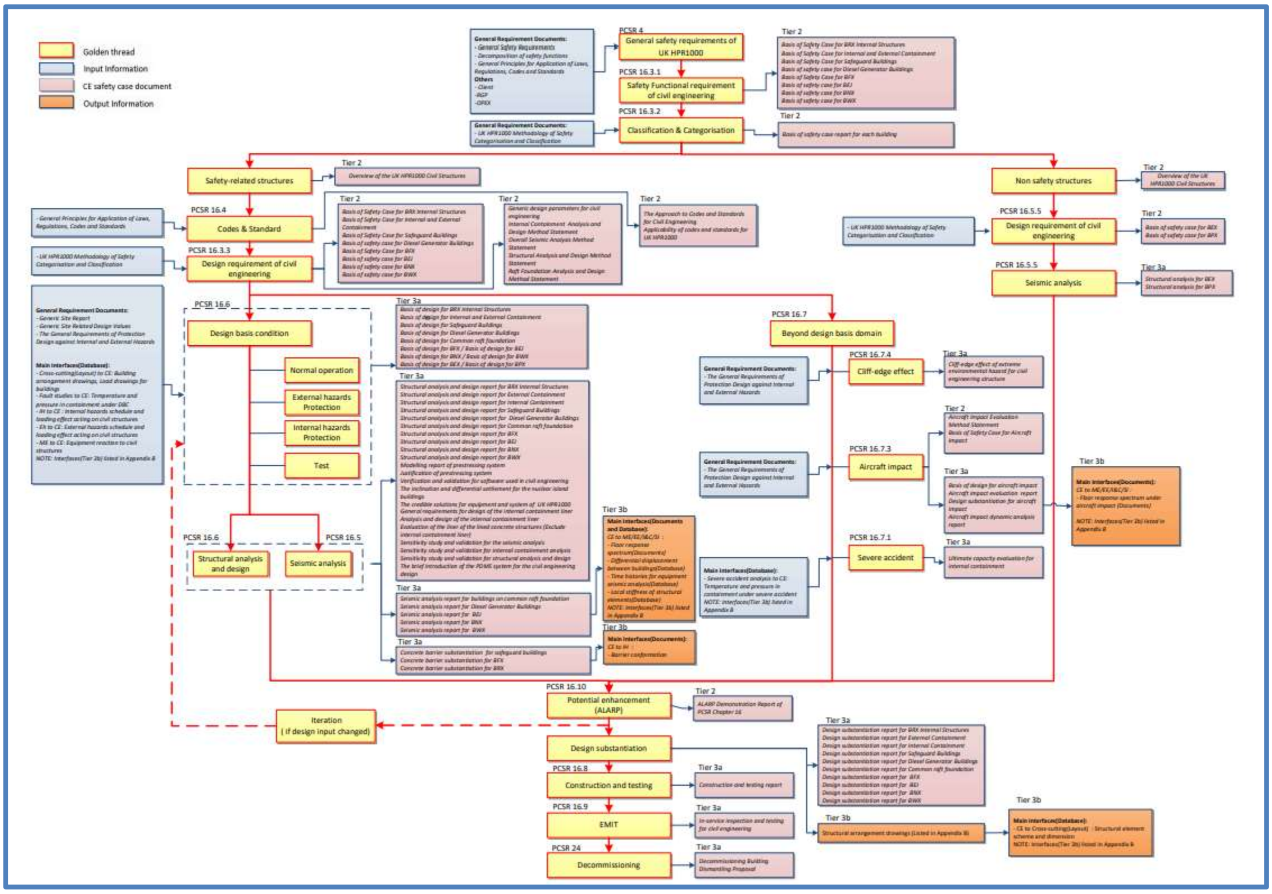

Figure 1: Golden thread of civil engineering safety cases. This diagram is presented here for illustrative purposes only as the text is not legible at this size. This diagram illustrates the golden thread shown in red arrows. This Figure is also included at full size in annex 6 in larger, accessible print. This diagram is extracted from Ref. *12* (noted therein as F-4-12)

63. Figure 1 shows that the golden thread of information through the documentation runs from the general safety requirements, to SFRs of civil engineering into classification and categorisation, with a link to safety related structures, codes and standards, to design requirements as inputs for the design. The process then considers design basis conditions of normal operation, external and internal hazards that input to the structural and seismic analysis and design stages. Figure 1 demonstrates how the ALARP consideration for potential enhancement feeds into the design substantiation reports and then into the PCSR Chapters (Refs. 3 and 13) for construction, EIMT and decommissioning.

64. The safety case is illustrated in an alternative manner in F-7-2 of Ref. 12, extracted as Figure 2 below. This figure categorises the various documents into whether they provide claims, arguments or evidence and illustrates the documents in that manner.

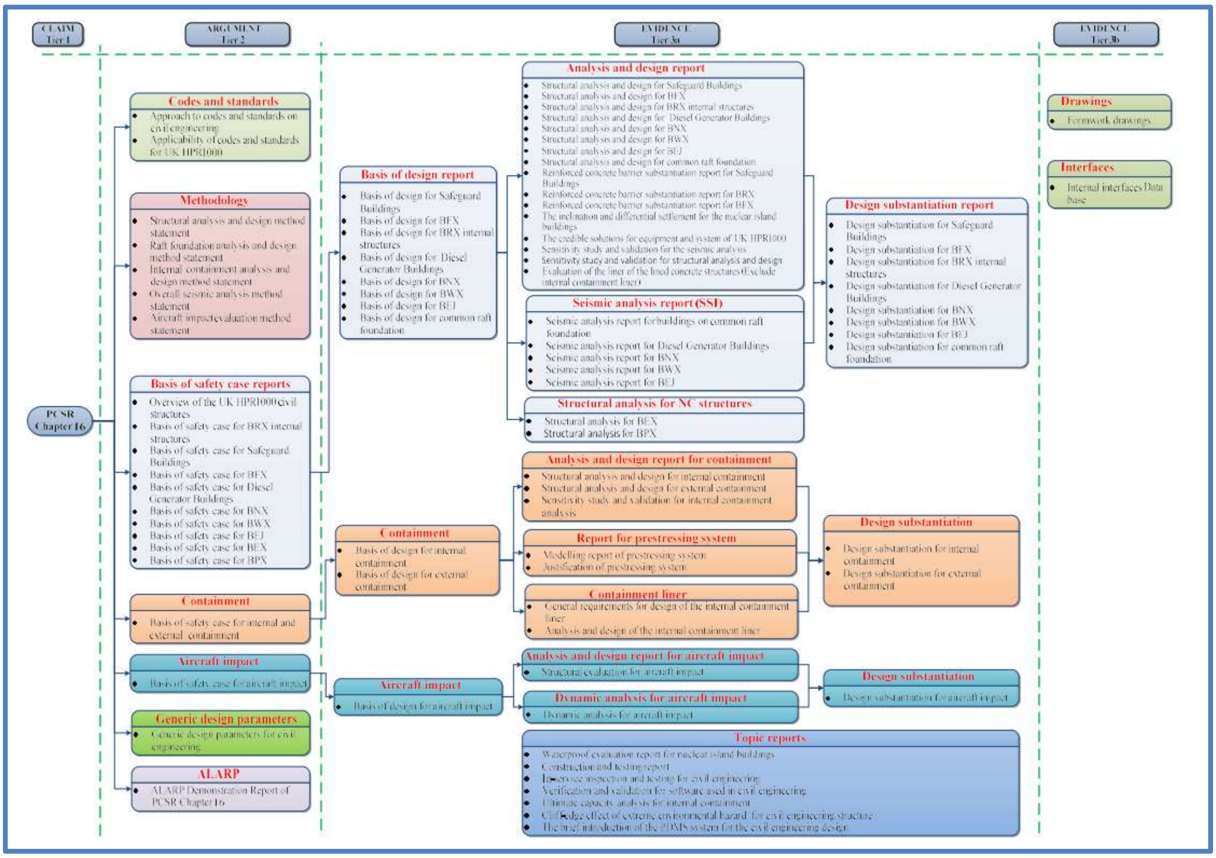

Figure 2: Documentation Map of Civil Engineering. This diagram is presented here for illustrative purposes only as the text is not legible at this size. This diagram illustrates the claims, arguments and evidence that the documents present. This Figure is also included at full size in annex 6 in larger, accessible print. This diagram is extracted from Ref. *12*, (noted therein as F-7-2).

65. The RP's documentation that explains the safety case presented for malicious aircraft impact is more complicated than Figure 2 illustrates. This topic interfaces closely with the external hazard discipline and reports for each discipline cross-reference each other. Figure F-2-2 of Ref. 14 diagrammatically describes the structure for this topic and an extract is included as Figure 3 below.

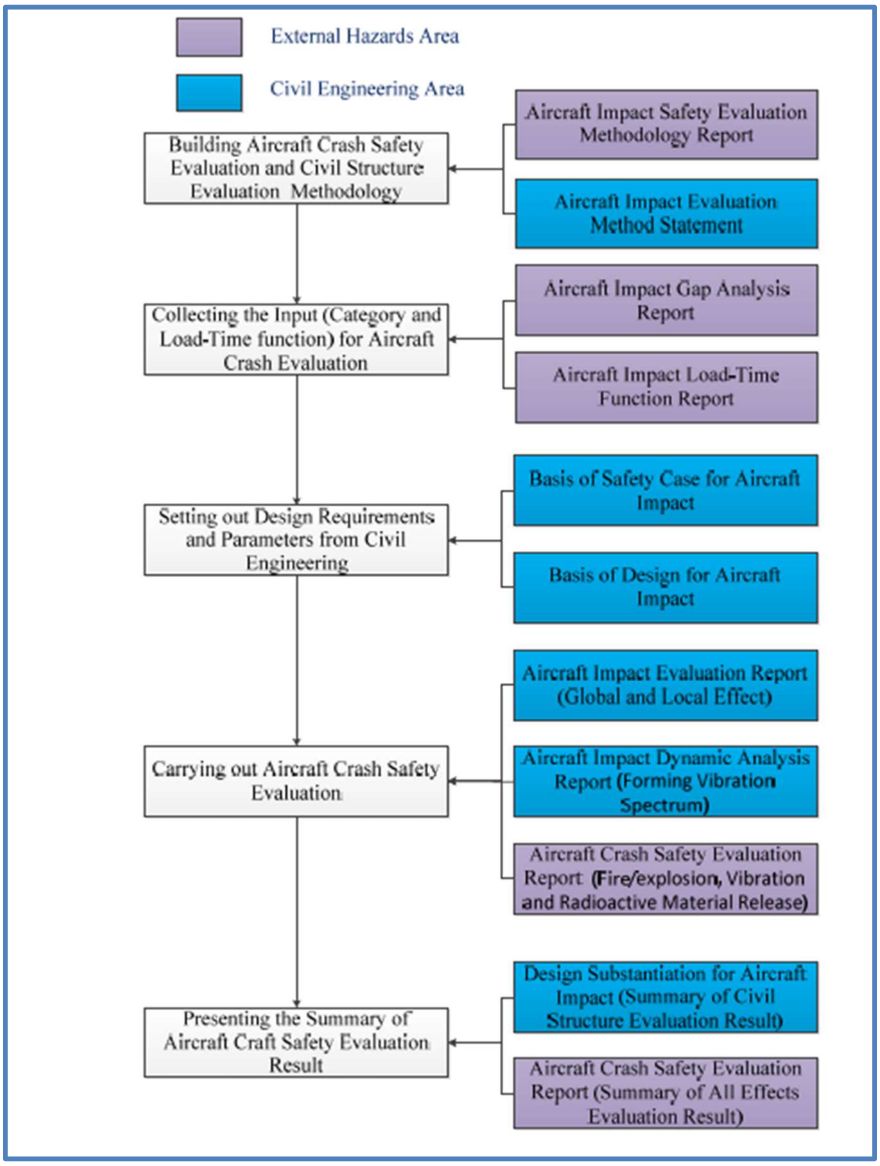

Figure 3: Documentation map for the aircraft impact safety case. The colours illustrate the interfaces and cross referencing between civil engineering and external hazards topic areas. This diagram is extracted from Ref. *14*, (noted therein as figure F-2-2).

3.2.2 Design Reference

66. The GDA design has developed incrementally over the GDA Step 4 assessment period. The RP has produced final safety case submissions based on Design Reference (DR) version 3.0, as described in the UK HPR1000 Design Reference Report (Ref. 15). However, it is normally the case for GDA that, due to the time and effort required to construct and run the analysis models for civil engineering, it is necessary to freeze the design at an early point. In this case, much of the RP's civil engineering modelling and analysis has been based on DR 1.0. Table 4 below confirms the design references that informed the analysis models assessed and identifies where the DR 1.0 was assessed and where DR 1.0 is unchanged for DR 3.0. I consider the implications of changes to the design beyond DR.1.0 in my assessment.

Table 4: Summary of design references used for structural analyses completed during GDA, (extracted from Ref. 12)

Structure	DR basis			
	Seismic analysis model	Detailed analysis model	Local analysis models and sensitivity models	AIA model
BFX	DR1.0	DR1.0	DR1.0 + DR3.0[1]	DR1.0 + AIA design change
Common Raft foundation	DR1.0	DR3.0	DR3.0	DR1.0 + AIA design change
IC[2]	DR1.0 (DR3.0)	DR1.0 (DR3.0)	DR1.0 (DR3.0)	DR1.0 (DR3.0)
EC[2]	DR1.0 (DR3.0)	DR1.0 (DR3.0)	DR1.0 (DR3.0)	DR1.0 (DR3.0)
Other buildings within GDA scope	DR1.0	DR1.0	DR1.0	DR1.0 + AIA design change

[1] For BFX, the DR used for the local analysis models including BFX -4.90m level floor and external walls with discontinuous thickness is DR1.0; the DR used for the seismic sensitivity analysis model defined in Section 4.2 of Ref. 16 is DR3.0.
[2] For internal containment and external containment, the DR1.0 and DR3.0 are the same.

3.2.3 Layout and design management

67. The layout and civil engineering design management for UK HPR1000 GDA has been based on the Fangchenggang nuclear power plant unit 3 (FCG3) 3D Plant Design Management System (PDMS) model.

68. During GDA, the RP claims to have developed a 3D PDMS ‘Improvement Model’ for use on the generic UK HPR1000 design. This ‘Improvement Model’ originated from the FCG3 3D PDMS model and the RP claims to have included design changes that have resulted from the design development throughout GDA Step 4.

69. The RP used the improvement model across all design disciplines, as a common, single source of information. The PDMS report (Ref. 17) explains the use of this 3D building information model in the context of civil engineering design. Therefore, layout requirements are implicitly captured within this multidisciplinary 3D PDMS model, rather than explicitly captured within documents.

70. It should be noted that the PDMS 3D model itself has not been submitted to ONR for assessment during GDA. However, extracts from the model have been used for illustrative purposes in document submissions and in meetings.

3.2.4 Overview of the main safety case documents

The principal documents are described in the following paragraphs.

<u>Pre-Construction Safety Report Chapter 16 (Ref. 3):</u>

71. The PCSR presents the safety case claims that the design, construction, operation and decommissioning of the UK HPR1000 are safe, secure and protect people and the environment.

72. The PCSR Chapter 16 (Ref. 3) uses the fundamental objective to form the high-level Claim 3, to derive Claim 3.3, which is then used to derive Claim 3.3.9. Claim 3.3.9 is then devolved into sub-claims specifically for civil structures. The PCSR then presents associated arguments and evidence for each sub-claim. The links between the claims,

sub claims and the documents that are presented for the arguments and evidence are illustrated in Figure 4.

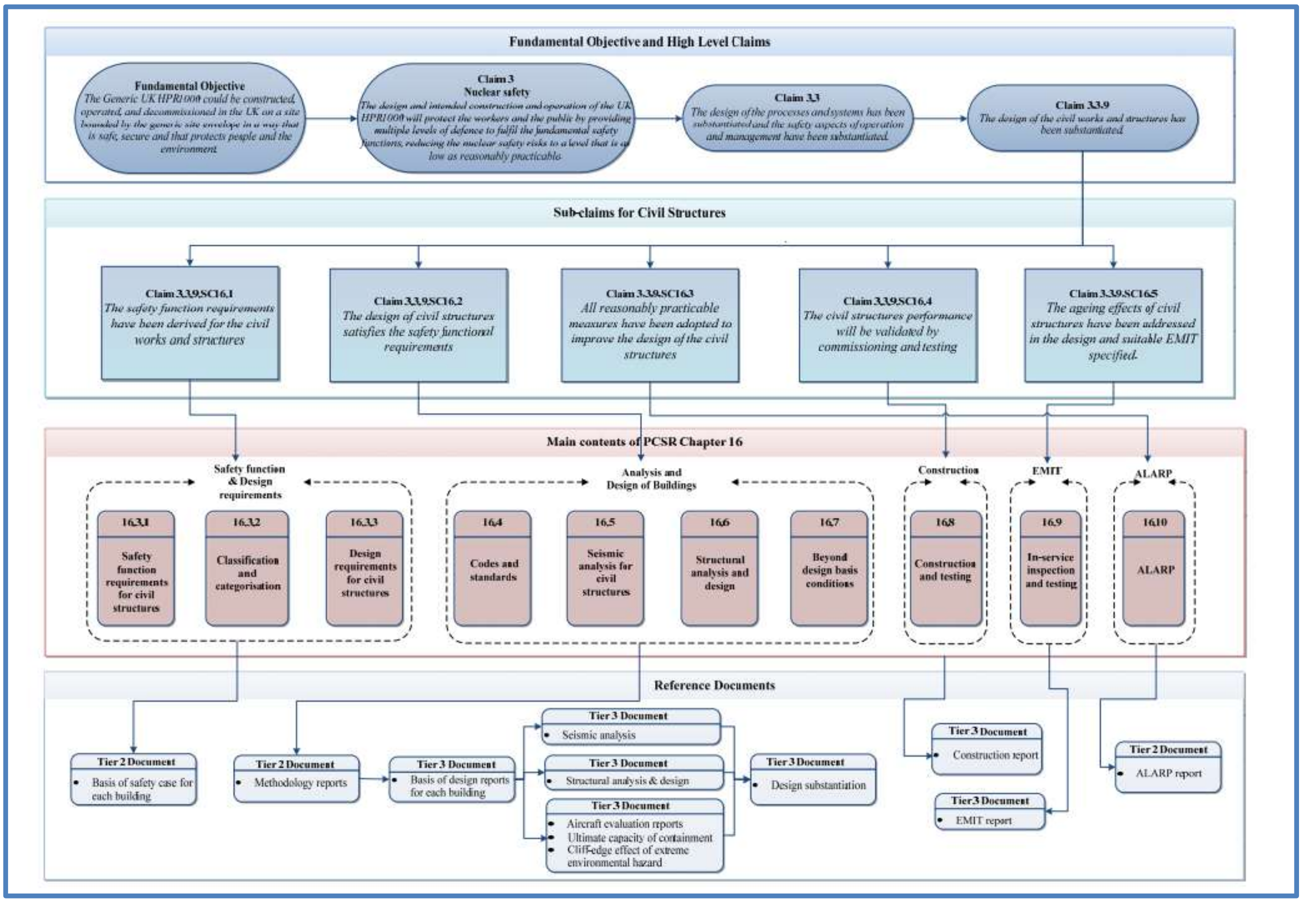

Figure 4: Structure of PCSR Chapter 16 – this diagram demonstrates the origin of claims from the fundamental objective to the high-level claims (Claim 3, 3.3 and 3.3.9) to the sub-claims for civil structures. This diagram is extracted from Ref. *3* (noted therein as F-16.2-4). This diagram is presented here for illustrative purposes only as the text is not legible at this size. This Figure is also included at full size in annex 6 in larger, accessible print.

73. The PCSR Chapter 16 (Ref. 3) highlights the golden thread for the civil engineering safety case and highlights the SFR schedules within the Basis of Safety Case documents. The PCSR describes the high-level safety functions that are relevant to civil engineering as:

- H4-2 (retention of water in fuel pool e.g., structural integrity to maintain heat removal),
- C3-2, C3-3, C5-1, C5-2, C5-3, C6-1, C6-3, (confinement),
- E1-4, E2-3 and E2-4 ('extra' functions).

74. The fundamental safety functional requirements and high-level safety functions are also described in the PCSR Chapter 4 (Ref. 18), alongside PCSR Chapter 16 Sub-chapter 4.4 (Ref. 3). The decomposition report (Ref. 19) derives the low-level safety functional requirements (SFRs) of civil structures (F1:F6):

- F1: Function to provide structural support to SSCs,
- F2: Function to provide protection to SSCs against external hazards,
- F3: Function to provide protection to SSCs against internal hazards,
- F4: Function to confine the radioactive materials, shield radiation, and reduce radioactive release,
- F5: Function to maintain specific internal building environments appropriate for SSCs during normal operating and accident situations,

- F6: Function to satisfy the requirements of decommissioning which include supporting SSCs, providing a barrier and confining the radioactive materials.

75. By means to demonstrate that the low-level SFR's (F1:F6) are achieved, structural behaviour requirements have been presented based on the safety functional requirements and classification and categorisation of the structures. These are referred to as design requirements (D1:D6):

- D1: Strength---Structures are strong and robust. Structures and structural members are designed to have required strengths at all sections,
- D2: Stability---Structures cannot collapse, and structures can maintain overall structural stability,
- D3: Durability---Structural durability can satisfy the design life requirement,
- D4: Serviceability---The deformation of the structure and structural members are limited and compatible with the requirements of equipment,
- D5: Water tightness---Structures are required to retain liquids,
- D6: Air tightness---Structures are required to contain gases under pressure with a limited rate of gas leaks.

76. The PCSR chapter 4 (Ref. 18) provides a high-level summary of the classification and categorisation for civil engineering structures, see Figure 5 below. Civil engineering structures are classified considering the highest classification and category of components housed within the building. The seismic category of structures is assigned depending on the required performance during and following a seismic event. Classification and categorisation of civil engineering structures are discussed in detail within the BoSC documents.

Building	Function Categorisation	Functional Classification	Seismic Category
BRX	FC1	F-SC1	SSE1
BSA/BSB/BSC	FC1	F-SC1	SSE1
BFX	FC1	F-SC1	SSE1
BNX	FC3	F-SC3	SSE1
BDA/BDB/BDC/ BDU/BDV	FC1	F-SC1	SSE1
BWX	FC3	F-SC3	SSE1
BEJ	FC3	F-SC3	SSE1
BEX	NC	NC	SSE2
BPX	NC	NC	SSE2

a) FC1: Category 1, FC3: Category 3, NC: Non-Categorised;

b) F-SC1: Functional Class 1, F-SC3: Functional Class 3, NC: Non-Classified;

c) SSE1: Seismic Category 1, SSE2: Seismic Category 2.

d) Function categorisation and functional classification of BDU/BDV are FC3 and F-SC3 respectively. Considering BDU is integrated with BDA/BDB and BDV is integrated with BDB, seismic category of BDU/BDV is same as BDA/BDB/BDC, i.e., SSE1.

Figure 5: Classification and Category of Structures. This Figure describes the functional classification and categorisation and the seismic category for the individual structures on site. This Figure is extracted from Ref. *12* (noted therein as Table T-16.3-1)

Codes and Standards:

77. With respect to codes and standards, the general approach adopted by the RP is to reference the principal US nuclear-specific codes and standards. However, in certain circumstances (one example being crack control) these codes are not deemed adequate by the RP and supplementary standards are used accordingly. For Aircraft Impact, guidance from IAEA and the US is applied as outlined in Section 3 of Ref. 20. The overall approach can be summarised as:

- Adopt American codes for the seismic analysis.
- Adopt American codes for strength (ultimate limit state) design, augmented with the use of:
  - European codes for material densities.
  - European codes and IAEA guides for fire conditions.
  - European codes for reinforcement splice and development lengths.
- Adopt a combination of American and European/ British codes for service ability (deflection, durability, crack control and water ingress protection) design.
- Plan for material specification in accordance with European and British codes.
- Adopt metric versions of codes and standards where available and generally process and store data in metric units.

78. This is covered by the following two documents

- The approach to codes and standards for civil engineering (Ref. 21)
- Applicability of codes and standards for UK HPR1000 (Ref. 22).

Ref. 21 presents the rationale for the codes and standards being used for civil engineering design. This report presents the outputs of the RP's workshops where the codes were compared and weighted against each other. Ref. 22 identifies areas for further consideration when using US codes in the UK context. It provides suggestions of the approach and methodologies to eliminate the gaps identified.

Design Principles and Methodologies:

79. These 'Method Statement' documents present the codes and standards, global and local analysis and design principles and methodologies. These cover the topics of 'overall seismic', 'structural', internal containment, aircraft impact, and common raft foundation as outlined below.

- Internal containment analysis and design method statement (Ref. 23)
- Overall seismic analysis method statement (Ref. 24)
- Structural analysis and design method statement (Ref. 25)
- Raft foundation analysis and design method statement (Ref. 26)
- Aircraft impact evaluation method statement (Ref. 27)

80. The RP's overall use of models and how they relate to the different analysis streams is illustrated in Figure 7 below. Where relevant to my assessment, aspects of these methodologies are described more fully in Section 4 below and further details can be found in Refs. 28 and 29.

81. These documents also outline the software utilised for modelling and finite element analysis. These are publicly commercially available codes that are applied extensively throughout the nuclear industry and are summarised as follows:

- ANSYS – Global structural static and dynamic analysis and internal containment analysis
- ANSYS LS DYNA – Non-linear dynamic analysis for malicious aircraft impact and internal hazard analysis.

- ABAQUS – Non-linear analysis of internal containment to evaluate its ultimate capacity
- ACS SASSI – Dynamic soil-structure-interaction analysis
- PDMS 3D – Management of spatial attributes of the design and layout
- Oasys Pdisp – Geotechnical analysis
- Rocscience SETTLE 3D – Geotechnical analysis

82. The RP has confirmed that, although the structural analysis and design of secondary steel structures are excluded from GDA scope, the software SAP2000 will be used for the detailed design and analysis post-GDA. Such secondary steel structures include access platforms needed for examination, inspection, maintenance and testing (EIMT).

83. The RP has chosen an internally developed custom software package for the design of reinforced concrete structural elements, specifically for the post processing of finite element geometric and load input data from finite element analysis. This software (REINCAL 1.0) produces design information such as areas of steel reinforcement, utilisation ratio values, crack widths and deflections. The verification and validation of this software is described in Ref. 30.

<u>Basis of Safety Case (BoSC):</u>

84. These reports listed below provide the safety case for each facility, the internal and external containment and the aircraft impact safety cases:

- BoSC for BFX (Ref. 31)
- BoSC for BRX internal structures (Ref. 32)
- BoSC for internal and external containment (Ref. 33)
- BoSC for safeguard buildings (Ref. 34)
- BoSC for diesel generator buildings (Ref. 35)
- BoSC for BEJ (Ref. 36)
- BoSC for BEX (Ref. 37)
- BoSC for BNX (Ref. 38)
- BoSC for BPX (Ref. 39)
- BoSC for BWX (Ref. 40)

85. Each BoSC presents a more detailed breakdown of the safety claims described in PCSR Chapter 16, and the methodology to derive civil engineering design from Safety Functions, aiming to clarify the link between civil engineering and upstream disciplines i.e., the faults and hazards (internal and external). These reports present the safety functional requirements (SFRs) and structural behaviour design requirements (namely D1:D6) applicable to each facility in order to satisfy the detailed functional requirements (F1:F6).

86. The appendices of these reports include the SFR schedules which are presented to link the SFRs to specific civil engineering Structures, Systems and Components (SSCs), with associated acceptance criteria, relevant design codes and standards and specific performance requirements. Helpfully, the SSCs referenced by the SFR schedules are also described and illustrated separately using 3D illustrations of the various structures and floor layouts extracted from the PDMS 3D model.

87. The BoSC reports do not cover explicitly the substantiation of barriers against internal hazards; rather the internal hazard reports need to be referred to for this information, see Ref. 41 and para. 93 below.

<u>Basis of Design (BoD):</u>

88. The reports listed below present the BoD for each structure, as well as the common raft, the internal and external containments and the aircraft impact protection.

- Generic design parameters for civil engineering (Ref. 42)
- UK HPR1000 Generic site report (Ref. 43)
- BoD for BFX (Ref. 44)
- BoD for diesel generator buildings (Ref. 45)
- BoD for BEJ (Ref. 46)
- BoD for BNX (Ref. 47)
- BoD for external containment (Ref. 48)
- BoD for BRX internal structures (Ref. 49)
- BoD for internal containment (Ref. 50)
- BoD for BWX (Ref. 51)
- BoD for safeguard buildings (Ref. 52)
- BoD for common raft foundation (Ref. 53)
- BoD for aircraft impact (Ref. 54)

89. The contents of these reports include the SFRs and design requirements, safety categorisations, applicable codes and standards, material properties and other specific design provisions (design loads and load combinations), and the procedures for decommissioning quality assurance and verification and acceptance criteria, with the aim to:

- Describe the requirements and provide sufficient information or reference to enable the analysis and design to be completed.
- Provide single source of information for designer.
- Link the engineering requirement and acceptance criteria (from Basis of Safety Case) to design information and/or parameters, including relevant codes and standards.
- Extract relevant information from Generic Design Parameters for Civil Engineering (Ref. 42), e.g., design life, external hazard information, material properties, etc.

90. These reports do not cover the basis of design for internal hazards barriers. Instead, this information is contained within the reinforced concrete barrier substantiation reports.

91. With respect to the derivation of the loads (or actions) and other design requirements originating from fault studies, external and internal hazards that are necessary for civil engineering design, these are communicated via hazard schedules within formal submissions separate to the civil engineering suite of documents. The BoD documents describe these inputs and their interpretation for use by civil engineering and, in some cases, provide cross references to the documents where the inputs are derived. For information on these documents please refer to Section 3 of the ONR assessment reports for External and Internal Hazards and Fault Studies (Refs 55, 56, 41).

92. Ref. 42 presents the definition and derivation of the generic site and design parameters used in the GDA civil structural design. This report, alongside the Generic Site Report (Ref. 43), is an important input to the Basis of Design reports. The parameters included are general assumptions; life span; reference temperature; geotechnical parameters; environmental data; and ground water. The external environmental parameters included are seismic response spectra; time histories; meteorological parameters such as air temperature, wind, snow, rain, etc.; hydrology; man made and industrial hazards. The two appendices in Ref. 42 provide further detail on the development of time histories and the derivation of input parameters for the soil-structure interaction modelling, thus clarifying aspects of the methodology.

<u>Reinforced Concrete Barrier Substantiation:</u>

93. The substantiation of the structural elements referred to as 'barriers' is covered in three Barrier Substantiation Reports (BSR) as follows:

- Reinforced concrete BSR for safeguard buildings (Ref. 57)
- Reinforced concrete BSR for BRX (Ref. 58)
- Reinforced concrete BSR for BFX (Ref. 59)

94. These reports span across the civil engineering and internal hazard disciplines, placing SFRs on the specific structural elements designated as ‘barriers’. The BSRs then present the substantiation of these barrier elements to the identified Internal Hazard loads and associated load combinations, which includes consideration of the internal flooding, fire, missiles, explosions, dropped loads and high energy pipe failures (HEPF). The BSRs present local and global element checks for the barriers, and where a load has a global effect, these loads are used as design input into loading combinations considered within the structural analysis and design reports. The results of these reports form part of the supporting evidence for the Design Substantiation Reports.

Seismic and Structural Analysis:

95. The seismic analysis for structures on the common raft foundation is reported by a single seismic analysis report, with a separate report for the structures on independent foundations away from the common raft. These reports are as follows:

- Seismic analysis report for buildings on common raft foundation (Ref. 60)
- Seismic analysis report for BNX (Ref. 61)

96. The seismic analysis is based on the parameters defined in the relevant BoDs, with a focus on structural behaviour under seismic demand with no direct links to SFRs. The reports include structural dynamic behaviour properties including frequencies, mode shapes, transfer functions and influence of soil. The main purpose of these reports is to calculate the seismic responses, including displacement and acceleration time histories, floor response spectra (FRS) and seismic demands to be applied to other models.

97. The structural analyses reports (SADRs) cover the BFX, internal containment and liner, the common raft, BNX aircraft impact and the internal containment liner as follows.

- SADR for BFX (Ref. 62)
- SADR for common raft foundation (Ref. 63)
- SADR for internal containment (Ref. 64)
- SADR for BNX (Ref. 65)
- Structural analysis for BEX (Ref. 66)
- Analysis and design of the internal containment liner (Ref. 67)

98. The SADRs draw information from the seismic analysis reports and the barrier substantiation reports (where global effects have been identified). The SADRs provide output from the analysis post-processing and design calculations, presenting the details of the entire analysis procedure, such as modelling, model loading, load case analysis, load combination, strength design and local analysis. The SADRs provide discussion of the results to provide understanding of structural behaviours including controlling load cases, thermal behaviour, conservatisms and construction and decommissioning considerations. The SADRs focus on structural behaviour and do not provide a direct link to SFRs (this is presented in the design substantiation reports).

99. The flow of information from and to the SADRs for the BFX facility is shown by Figure 6 below.

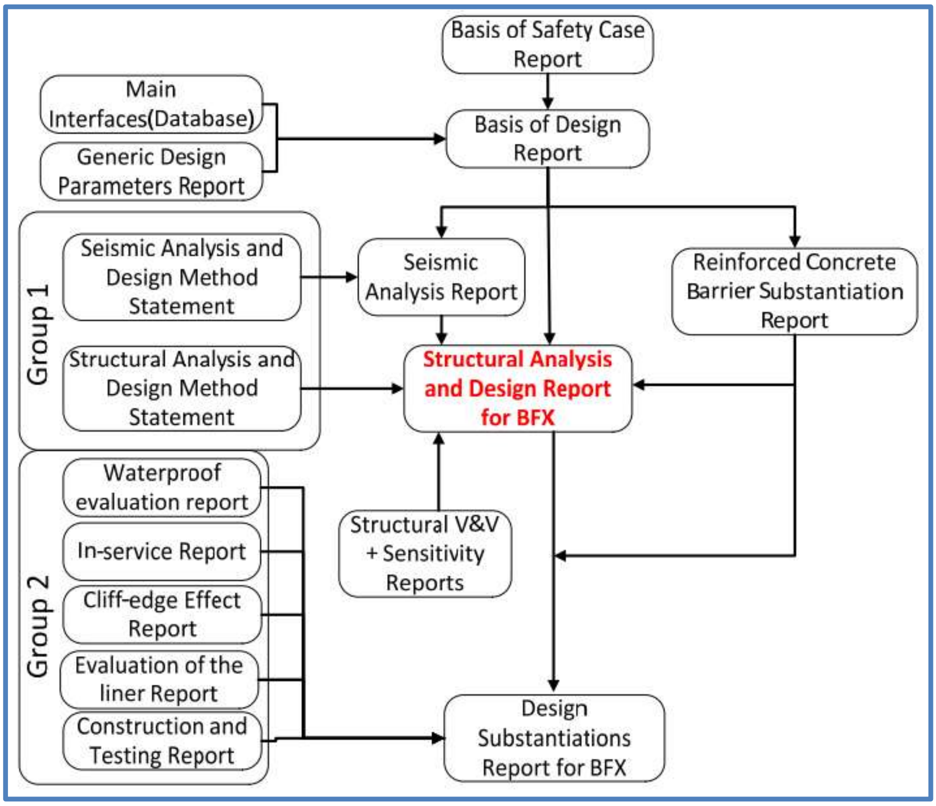

Figure 6: Document interface and report hierarchy. Diagram demonstrating the inputs to the SADR for BFX (Ref. *62*). Diagram extracted from Ref. *12* (noted therein as F-2.2-2)

Structural Verification and Validation and Sensitivity:

100. This topic area is covered by three reports as follows:

- Sensitivity study and validation for the seismic analysis (Ref. 68)
- Sensitivity study and validation for internal containment analysis (Ref. 69)
- Sensitivity study and validation for structural analysis and design (Ref 70)

101. These three reports present the verification[5], validation[6] and sensitivity studies for the seismic, general structural, and internal containment analyses and provide discussion on the findings. These reports link into and support the SADRs and DSRs. Details of the verification and validation are discussed in Section 4, with added detail contained in Refs. 28 and 29.

Other topics:

102. The RP has produced 'Group 2' reports that present the supporting evidence on a range of generic (not building-specific) civil engineering topics (some of which are shown in Figure 6, above), namely:

- Waterproof evaluation report for nuclear island buildings (Ref. 71)
- In-service inspection and testing for civil engineering (Ref. 72)
- Cliff edge effect of extreme environmental hazard for civil engineering structure (Ref. 73)

[5] Verification here refers to the process of determining that a computational model accurately represents the underlying mathematical model and is capable of reproducing its theoretical solution.
[6] Validation here refers to the process of determining the degree to which the model (including the parameters selected for that model) is an accurate representation of the real world from the perspective of the intended uses of the model.

- Verification and validation for software used in civil engineering (Ref. 30)
- Ultimate capacity analysis for internal containment (Ref. 74)
- Construction and testing report (Ref. 75)
- The introduction of the PDMS system for the civil engineering design (Ref. 17)

103. The Group 2 reports present the substantiation of aspects of the GDA design to justify the safety case for these topics, to support the claims, arguments and evidence that are presented in the Design Substantiation Reports. These Group 2 topic reports reference out to supplementary reports which support the SADMS and the SADRs, namely:

- The inclination and differential settlement for nuclear island buildings report (Ref. 76)
- The credible solutions for equipment and systems of UK HPR1000 report (Ref. 77)
- The sensitivity studies and validation reports (see para. 100 above)
- Construction and decommissioning reports (Refs 78, 75 and 79)
- Reports on the internal containment prestressing and liner (Ref. 80, 81, 82, 83 and 67)

Design Substantiation:

104. The RP has produced the following Design Substantiation Reports (DSRs) that cover each facility, the internal and external containment and the aircraft impact case.

- DSR for Common Raft Foundation (Ref. 84)
- DSR for BFX (Ref. 85)
- DSR for Internal Containment (Ref. 86)
- DSR for BNX (Ref. 87)
- DSR for Aircraft Impact (Ref. 88)

105. These reports present the design substantiation to demonstrate that all the safety functional requirements can be achieved, presenting the SFR schedules defined in the BoSCs and linking together the evidence provided in other reports outlined above and documents from other disciplines (e.g., fire strategy report).

ALARP demonstration:

106. In the Guidance for Requesting Parties (Ref. 1), there is a fundamental GDA requirement for the RP to set out their process to reduce the risks from the operation of the reactor design to a level that is As Low As Reasonably Practicable (ALARP). For civil engineering this is presented in Ref. 89. This report presents underpinning to the Sub-chapter 16.10 of PCSR Chapter 16 (Ref. 3), aiming to support the Sub-claim 3.3.9.SC16.3, which is: *"All reasonably practicable measures have been adopted to improve the design of the civil structures"* in line with the ALARP methodology document.

Drawings and Databases:

107. Drawings and the RP's internal interfaces database are presented where appropriate for further lines of inquiry. A full list of documents is provided in Appendix B of the production strategy (Ref. 12). Examples of these documents include:

- Floor response spectrum documents
- Differential displacement between buildings database
- Time histories for equipment seismic analysis database
- Local stiffness of structural elements database
- Drawings (for examples see Refs. 90, 91, 92, 93, 94, 95, 96, 97, 98, 99)

108. There are requirement management schedules that confirm the RP's forward commitments that will be undertaken at the site-specific phase. The commitment capture log (Ref. 100) presents this information for civil engineering.

4 ONR ASSESSMENT

109. The reporting structure of this section follows the assessment sampling strategy outlined in Section 2. This is augmented by three further sections; firstly to provide an overview of the RO's applicable to this assessment; secondly to provide a judgement regarding whether the GDA work has sufficiently demonstrated that risks arising from civil engineering have been reduced so far as is reasonably practicable (SFAIRP); thirdly, due to the number of regulatory queries raised during my assessment, the final section considers whether the RP has adequately consolidated the technical material into the suite of civil engineering safety case documentation presented by PCSR Chapter 16 (Ref. 3).

110. Therefore, the sections of the assessment follow the structure below.

- Output from previous GDA steps
- Regulatory Observations and Regulatory Queries
- Civil engineering safety case
- Design principles and methods for reinforced concrete primary structures
- Application of design principles and methods to:
  - Sample 1 – BFX (SSE1 structure on common raft)
  - Sample 2 – Internal containment
  - Sample 3 – Common raft foundation
  - Sample 4 – BNX and BDB/BDV (SSE1 structures on individual rafts)
  - Sample 5 – BEX (SSE2 structure)
  - Sample 6 – Malicious aircraft impact protection
- Further safety case considerations
- Demonstration that relevant risks have been reduced to ALARP
- Consolidation of safety case within PCSR Chapter 16
- Comparison with standards, guidance and RGP

111. The strengths, outcomes and conclusions are highlighted at the end of each section. In accordance with ONR guidance (Ref. 1), assessment findings are raised in the assessment text denoted 'AF-UKHPR1000-XXXX'. A summary of these findings is provided in Annex 4.

112. For civil engineering, the GDA process has limitations as outlined in Section 2.2.2. Therefore, the judgements formed in this report are bespoke to the GDA process and for the civil engineering discipline. Therefore, when the phrase "for GDA", or "for the purpose of GDA" is used within this assessment report, the commentary in Section 2.2.2 should be borne in mind.

113. As described in Section 2.5, my assessment was supported by a TSC team of experts. This team provided expert advice on in-depth technical topics, such as:

- Finite element modelling and analysis methodologies
- Use of non-linear modelling and analysis methods for the evaluation of the internal containment ultimate capacity
- Use of non-linear and empirical analysis methods for substantiating against internal hazards
- Dynamic and static SSI analysis
- Concrete strength design and evaluation of in-house software ReinCal
- Development of fragility curves for the internal containment
- Use of non-linear finite element and empirical analysis methods for validating and substantiating structures against aircraft impact analysis
- Verification and validation strategy for modelling and analysis

114. The scope of the TSC technical review is recorded in Refs. 28 and 29. The structure of the TSC reports was deliberately designed to align with the structure used by this

report to enable the reader to access background and further technical detail efficiently. I make reference to these reports in my assessment where applicable to a judgement or decision. The regulatory judgment on the adequacy, or otherwise, of the generic UK HPR1000 safety case in this report has been made exclusively by me, informed where appropriate by the TSC's expert advice.

115. As outlined in Section 2.2.1, my assessment of the RP's formally submitted extant safety case was further enabled via technical workshops with the purpose of further information gathering; see paragraph 34. Each workshop was focused on a specific topic area (see Table 1) and enabled the RP's technical team to demonstrate their understanding, technical competence, and capability. The workshops enabled me to pose queries regarding the detailed technical aspects of the RP's technical presentations. All technical queries from a workshop that were not adequately covered by the technical presentations were then formalised within regulatory queries. This ensured queries were progressed, and additional technical material and commitments to further work were incorporated appropriately into safety case documentation. Some of these Regulatory Queries gave rise to additional technical workstreams by the RP; examples being the analysis of the internal containment gusset region, justification of the RP's thermal analysis approach, aspects of the aircraft impact analysis, and walkthroughs of the analysis and design process for sample members. Engagement on these technical topics and other Regulatory Queries were progressed via a series of Regulatory Query technical progress meetings (see Table 2). The discussions in these meetings resulted in further Regulatory Queries being raised on specific technical details, as necessary.

116. With respect to Regulatory Observations, my GDA Step 4 assessment did not result in any new Regulatory Observations being raised. Further, where necessary, I have provided civil engineering technical input to other Regulatory Observations that have been raised by other ONR disciplines. This is discussed in Section 4.2 below.

4.1 Output from Previous GDA Steps

117. The RO's originating from previous GDA steps are discussed in Section 4.2 below.

118. The GDA Step 3 civil engineering assessment (Ref. 9) raised 33no. *'Open Points'* and 20no. *'Areas for Improvement'*. All of the *'Areas for Improvement'* and some of the *'Open Points'* were captured by RQ's and are described in Annex 3 below, and tables 1 & 2 of Ref. 9. These RQ's (described in Annex 3) were satisfactorily resolved during my Step 4 assessment and are considered adequately closed. Those *'Open Points'* not captured and resolved by RQ's (see Table 2 of Ref. 9) were covered within my Step 4 assessment scope. Table 5 below articulates the relevant sections of this assessment where these topics are assessed.

Table 5: Summary of the assessment of Step 3 'Open Points' during this assessment

Open Point No. (see Ref. 9)	Description	Relevant section of this report
OP-3	The use of the PDMS model to manage the spatial configuration and layout should be assessed specifically during Step 4.	§4.3
OP-4	The substantiation of the Common Raft should be looked at specifically during Step 4, with the following areas noted: • the adequacy of the raft thickness and stiffness. • the suitability of the raft design scheme for the GDA range of ground conditions (fulfilment of RO-UKHPR1000-0009). • the location and configuration of the transition in raft thickness. • the monolithic pre-stressing gallery walls and raft interface.	§4.7

Open Point No. (see Ref. 9)	Description	Relevant section of this report
OP-5	I recommend that the analytical modelling, design details and substantiation for elements located below the seismic isolation joints that cross the Common Raft (see red lines on Figure 4) are sampled in Step 4 to ensure the modelling accurately reflects the design intent.	§4.4
OP-7	Further assessment of the application of the RP's methodology to deeply embedded structures (including derivation of appropriate inputs) should be assessed further during Step 4.	§4.8
OP-8	Further assessment regarding the modelling of 1-D beam and column elements and how they interface with 2-D shell elements is needed during Step 4.	§4.4
OP-13	I recommend a more detailed assessment of the IC liner (involving structural integrity as necessary) is carried out once the reports are delivered during Step 4.	§4.64.6.8
OP-19	The RP's method inferred from ACI349 for calculating the reinforcement demand in 2-D concrete elements is yet to be demonstrated reliable for all conceivable and/or demonstrated stress states. This will be taken forward collaboratively in Step 4 via engagement with external experts and stakeholders.	§4.4
OP-20	Further assessment of the methodologies is needed in Step 4 for SSC's where deflections may be critical e.g., Crane supports and service penetrations	§4.6
OP-24	With respect to aircraft impact further information is expected in Step 4 regarding the damping used and its justification in different parts of the structure. If the analysis shows that parts remain lightly stressed, then low values of damping should be used.	§4.10
OP-25	With respect to aircraft impact, appropriate justification will need to be provided in Step 4 if the RP intends to assume that some equipment can withstand scabbing loads. Otherwise, the RP should design wall thicknesses to prevent scabbing, or assume that equipment behind scabbed walls is rendered unusable.	§4.10
OP-26	The approach to justify the beyond design basis is yet to be documented fully for assessment. Further assessment is needed during Step 4.	§4.4
OP-28	I recommend a more detailed assessment of the pool liners (involving structural integrity as necessary) is carried out once the reports are delivered during Step 4.	§4.5
OP-30	Further assessment is recommended to look at the BRX polar crane support system during Step 4.	§4.6
OP-31	Further assessment is required during Step 4 of the design process described within the Barrier Substantiation Reports and the information provided in RQ-UKHPR1000-0510 (Ref. 6).	§4.4 & §4.5

OP-32	Further detail and assessment is needed during Step 4 regarding the following areas;
	• Details of the seismic joints including
	  ○ the SFRs and DRs for the seismic joints,
	  ○ the methodology to define the magnitude and direction of differential movement (e.g., to safeguard against seismic pounding and/or shearing of seals and bridging services due to settlement);
	  ○ the assurance of watertightness across joints.
	• The civil engineering aspects of the FTT including the SFRs and the detailed design requirements such as the movement range and design details.
	• The adequacy of the BFX roof section size and construction details.
		§4.3, §4.4, §4.5, §4.6, §4.8 and §4.9

OP-33	Further assurance is needed in Step 4 to show that conventional (CDM) hazards, including those associated with or affected by EMIT requirements, are influencing design decisions within the Civil Engineering discipline. The areas of follow up in Step 4 to assess these constructability aspects and the application of CDM in collaboration with conventional H&S specialists are: • BFX roof, • EC dome • Common raft & gusset zone in the vicinity of the IC & EC walls	§4.11

4.2 Regulatory Observations & Regulatory Queries

119. The one civil engineering RO (RO-UKHPR1000-0009) that originated from GDA Step 2 has been closed, see Ref. 101. RO's from previous GDA steps raised and led by other ONR disciplines that required civil engineering input have also been closed. The civil engineering input to these during Step 4 are summarised in Table 6 below.

120. RO-UKHPR1000-0009 is discussed extensively in Section 4.7.4 and Ref. 101. The table below summarises those ROs that I have supported, the topic lead for those ROs, the input I provided, and the location of the assessment detail. The reader should refer to the relevant RO assessment notes and/or the related ONR assessment report.

Table 6: Summary of RO's relevant to this civil engineering assessment

RO Number and title (see Ref. 7)	Topic Lead	Civil engineering input & RO status
RO-UKHPR1000-0002: Demonstration that the UK HPR1000 Design is Suitably Aligned with the Generic Site Envelope	External Hazards	Input provided on the overall adequacy of the GDA substantiation of civil structures. See the RO closure note (Ref. 102) and Section 4.18.1.1 of the External Hazards assessment report (Ref. 55). RO closed.
RO-UKHPR1000-0004: Development of a Suitable and Sufficient Safety Case	Cross Cutting	Input provided on the adequacy of the safety case golden thread from and to civil engineering, the development and traceability of SFRs, and the framework for the SFR schedules. See Section 4.3 for relevant assessment commentary and the cross cutting ONR assessment report that summarises closure (Ref. 103). RO closed.
RO-UKHPR1000-0007: Aircraft Impact Safety Case for UK HPR1000	External Hazards	Input provided on the adequacy of the GDA analysis methodologies and substantiation for the civil engineering structures. See Section 4.10 below, the RO closure note (Ref. 104), and Section 4.18.1.2 of the External Hazards assessment report (Ref. 55). RO closed.
RO-UKHPR1000-0008: Justification of the Structural Integrity Classification of the Main Coolant Loop	Structural Integrity	Advice sent to structural integrity to support closure of Actions 2 & 3, see RO closure note (Ref. 105) and the Structural Integrity assessment report (Ref. 106). RO closed.
RO-UKHPR1000-0009: Geotechnical Design Parameters	Civil Engineering	This was assessed by civil engineering with input from External Hazards for Action 1. See Section 4.7.4 below and the RO closure note (Ref. 101). RO closed.
RO-UKHPR1000-0014: Spent Fuel Building – Design of Nuclear Lifting Operations to Demonstrate Relevant Risks are Reduced to ALARP	Mechanical Engineering	Advice provided on the structural modification proposals for BFX. See Section 4.5.7 below and the RO closure note (Ref. 107). RO closed.
RO-UKHPR1000-0054: Validation of Internal Hazard loadings used for civil engineering design of non-barrier elements	Internal Hazards	Development of decoupling approach for internal hazards, and improvements to SFR schedules for non-barrier structural elements see Sections 4.4.2 and 4.5.5 below, the Internal hazards assessment report (Ref. 41) and the RO closure note (Ref. 108). RO closed.
RO-UKHPR1000-0056: Fuel Route Safety Case	Cross Cutting	Advice provided on the structural modification proposals for BFX. The assessment of the RP's gap analysis report (Ref. 16) revised for the resolution of this RO is recorded in Section 4.5.7 below. Also refer to the RO closure note (Ref. 109). RO closed.
RO-UKHPR1000-0058: Classification of Main Steam Line and Associated Major Valves in Safeguard Buildings	Structural Integrity	Review of civil engineering aspects of the RP's consequences and ALARP assessments. Advice provided to structural integrity discipline, see RO closure note (Ref. 110) the Structural Integrity assessment report (Ref. 106). RO closed.

121. As noted in paragraph 115, RQ's were used to formalise and record technical queries during Step 4. Furthermore, these RQ's subsumed the residual matters that arose from Step 3. In total, over 200 RQ's were raised during my assessment and these are summarised in Annex 3 below.

4.3 Civil Engineering Safety Case

4.3.1 Scope and Limitations

122. The GDA scope for the RP's civil engineering safety case was confirmed at the outset of Step 4 and formally recorded by the RP's Production Strategy document (Ref. 12). From my assessment, I am satisfied that the RP has delivered the intent of this scope and this is demonstrated by the presentation of their civil engineering safety case in the set of formal submissions described in Section 3.

123. The civil engineering safety case does not consider, and therefore does not develop SFRs and associated design information, for structures that surround and interface with the GDA structures. This is consistent with the limitations and exclusions identified in Section 2. The safety case will need to be augmented to cover this fully as part of the future site-specific phase. I consider this normal business.

4.3.2 Safety Case Approach and Structure

124. With respect to the safety case documentation structure described in Section 3, and the expectations of SAP SC.4, my assessment is based on the sample areas described in Section 2. For these sample areas, my assessment specifically focussed on the golden thread and corresponding content across the relevant civil engineering reports and related cross cutting reports that feed into civil engineering. From my assessment, I am satisfied that the RP's structure and approach to the safety case reflects the claims, argument, evidence approach. The PCSR Chapter 16 (Ref. 3) acts as the hierarchical document, referencing out to civil engineering documents. For the purpose of GDA, I am content that the civil engineering safety case document structure sufficiently meets the intent of SAP SC.4, albeit I expect improvements to be made for the site-specific phase.

125. An area for improvement evident from my assessment pertains to the flow of information and level of detail within the civil engineering safety case. I note that PCSR Chapter 16 (Ref. 3), alongside the decomposition of safety functions report (Ref. 19), identifies the decomposition of the lower-level safety functions from the high-level safety functions. For the sample areas identified in Section 2, I have traced the links from the low-level safety functions to the SFRs; presented in the BoD report. For the internal containment, I noted that the level of detail was adequate in the SFR schedules, BoDs, BoSCs, DSRs and other references, where the golden thread of civil engineering design requirements through to substantiation could be followed. This was similarly the case for the BFX and common raft structures. From my assessment of other structures, I found differing levels of detail and maturity in reports, and this is an area where the RP will need further work to bring all reports to a consistent level of detail.

126. Furthermore, I identified occasions where the method statements contain design parameters or details of the solutions which would be better located in other reports, for example Ref. 25 states a width for seismic joints which is irrelevant to the analysis and design methodology. I have noted the SADRs originated in advance of the BoSCs and BoDs being fully developed, which could be a contributing factor to SADRs containing unnecessary repetition of information from the BoDs and the method statements, whilst also containing information that might be better located in the upstream documents.

127. From my assessment, there is a clear need to further develop the civil engineering safety case for all structures to ensure that all reports are brought up to a consistent level of detail and maturity, including cross over reports that interface with other disciplines. For GDA I judge this to be a minor shortfall and am content that this further development of the civil engineering safety case to fully meet the intent of the SC-series of SAPs will form part of normal business in future phases of design development.

4.3.3 Layout and Design Configuration

128. The generic UK HPR1000 design has been derived from the design of Fangchenggang nuclear power plant unit 3 which is under construction. It is evident that the RP has a history of design experience through the development and deployment of this design in China. For civil engineering, the GDA design is not a development of a conceptual design that is being optimised from first principles. Rather, it is the re-substantiation of an existing design (that is partway through construction in China) against RGP, evaluated in accordance with the UK's goal setting regulatory regime.

129. The principal purpose of the civil engineering design is to meet the safety functions to maintain the safety of the plant operations. For this GDA, as the generic UK HPR1000 safety case information has been developed through the GDA, the SFRs have essentially been backfitted to the existing design rather than directing the design. However, I am content that this is unavoidable given the background described above.

130. The majority of the layout decisions and requirements are made upstream of the civil engineering design and are recorded within a PDMS 3D system or model. I note that, whilst this 3D model was not in itself submitted to ONR as formal evidence to support safety claims, drawings and extracts from the model were provided by the RP during Step 4 in formal submissions and have been assessed accordingly and provided assurance with respect to the RP's processes. From my assessment of the RP's requirements for the civil engineering layout in PCSR Chapter 16 (Ref. 3) and PSCR Chapter 2 (Ref. 111) I note they are high level and cover shielding, access, and protection from hazards (internal and external). Nonetheless, from my assessment I am content for GDA that layout requirements are adequately transmitted and reflected in the civil engineering safety case documentation. From a civil engineering perspective, I consider the use of the PDMS 3D system in this manner to have many advantages, and I am content that this represents RGP and can be assessed further as normal business in future phases. For the broader cross-cutting assessment of layout requirements, the reader should refer to ONR's assessment of RO-UKHPR1000-0004 recorded in Ref. 103.

131. It is noteworthy that the geometries of the civil engineering design, as defined by the plant layout, are critical to the substantiation of the civil engineering design against the relevant safety functions. Additionally, that civil engineering design decisions can impact whether the design is considered to have reduced risks so far as is reasonably practicable (SFAIRP), which in some instances would then feed-back into layout decisions. I have assessed examples of this during my Step 4 assessment, e.g., the fuel transfer tube and the interface between foundation design and services design. These examples considered the civil engineering design layout, which I judge demonstrated a robust logic behind the engineering decisions that resulted in reduction of risks in line with the ALARP principles. Additionally, in workshop #12, (see Table 1 above and Ref. 10), the RP presented the optioneering process around the design modifications e.g., BFX geometry and common raft thickness. These examples demonstrated that the process used in making decisions when choosing options for design modification had examined the merits of the options available. The RP considering safety and other factors which were scored, to inform decisions, which is in

line with the expectation that decisions are informed by a process that considers the relative merits of available options.

132. With respect to the design management and Technical Change Note (TCN) control process (Ref. 112), this is assessed in the Management for Safety and Quality Assurance Assessment Report (Ref. 113). During my assessment, I have noted examples of this TCN process being implemented e.g., M60 for the changes to the common raft thickness, M27 and M81 for the increases of wall thickness associated with malicious aircraft impact protection and M94 for the BFX geometry and layout configuration changes (the reference documents are recorded in the GDA Step 4 modifications log at Ref. 114). Through these examples, I saw the output of the management arrangements for making decisions around layout changes. The civil engineering design team worked alongside other disciplines, feeding into a central layout team who manage the 3-D Plant Design Management System (PDMS) improvement model coordination to implement proposed design changes and the update of the design reference information. I consider that the RP's use of the PDMS 'improvement model' (see Section 3.2.3) is appropriate as a useful communication tool, promoting common understanding, visualisation and illustrating the way that structures, systems and components interact. From my assessment, I am satisfied that the use of this PDMS improvement model as a single source of information, alongside the TCN change control process and version control that is managed by a centralised layout team is adequate for the purpose of GDA. I am content that the interface controls for civil engineering design management are suitable for the complexity of the civil engineering design at this stage in GDA and they provide a framework for the site-specific development of the design.

133. A caveat to the above, related to Section 3.2.2, is that the evolution of the design during GDA Step 4 has resulted in a situation where the PDMS model has been updated, but some aspects of the civil engineering analysis are based on a previous version of the layout. I recognise that parallel working across disciplines necessitates a design freeze to enable analysis to be completed as part of the design works. Therefore, I consider the RP's approach to this of undertaking civil engineering analysis based on DR1.0 to be a reasonable and pragmatic way of accommodating all the disciplines working in parallel. However, this is an important limitation that requires further work to harmonise the civil engineering analysis and design with the latest design reference. This requirement for further work is captured by assessment finding AF-UKHPR1000-0214.

AF-UKHPR1000-0214 – The licensee shall, as part of detailed design, ensure that the structural analysis models and design information for civil engineering is harmonised consistent with the latest design reference.

134. Related to this, the Design Reference Report (Ref. 15) provides dimensioned drawings that reference changes that are proposed for GDA. Although I consider these sufficient for the purpose of GDA, these drawings do not provide the level of detail required to understand the location of the structures relative to adjacent buildings. Improvements to site wide layout drawings and structure proximity will be needed in the site-specific phase, I consider this to be normal business.

135. In summary, my assessment has not identified any civil engineering aspects of the design layout that would preclude the RP from demonstrating their layout would reduce risks so far as is reasonably practicable during the site-specific phase and meeting the intent of SAPs ELO.1 and ELO.4.

4.3.4 Inputs – Internal Hazards and External Hazards

136. I have reviewed the hazards relevant to civil engineering from the general requirements report (Ref. 115) and I note that 'high integrity components' (HIC) are screened out based on credibility of failure, and volcanoes and meteorites have been screened out on a frequency basis. All other hazards are implicitly screened in and considered for design basis and, where applicable, severe accident analysis. I am content that the screening process for internal and external hazards that impact civil engineering design has been adequately undertaken and that these examples identified are in line with the intent of SAP EHA.4 and FA.5. I have based this conclusion on both my assessment and communication with the ONR GDA assessors undertaking the assessment of internal and external hazards. For further assessment of identification of fault barriers for fault sequence termination, hazard screening, intensity definition, fault and hazard protection schedules and radiological shielding requirements, in accordance with the SAPs EHA-series, see the internal and external hazards and fault studies assessment reports (Refs. 41, 55, 56).

137. Some external hazards[7] have yet to be assessed as these require site-specific parameters. I am satisfied that this approach to the UK HPR1000 GDA meets the intent of SAP SC.4, as the site-specific hazards will be assessed at the site-specific phase, which I consider to be normal business. Following confirmation with the ONR External Hazards Inspector, I am satisfied with the screening and derivation of external hazards associated with civil engineering, the communication of which has been through the external hazards schedule report (Ref. 116). For the assessment and consideration of minor inconsistencies, see the ONR external hazards assessment report (Ref. 55).

138. The derivation of the internal hazard loads[8] has introduced specific challenges to the civil engineering design. In order to allow the RP's civil engineering and internal hazards safety case submissions to proceed in an optimised manner, a decoupling strategy was adopted, whereby conservative loads were defined by expert judgement for use in design. For the background context, please refer to the internal hazard assessment report (Ref. 41). From a safety case perspective, I am content that this approach is adequately conservative and reported appropriately to allow the purposes of GDA to be met. Nonetheless there are some important areas of future work that are discussed further in Section 4.4.2 below.

139. I have reviewed the internal and external hazard schedules (Refs 117 and 116) and I note that they list the hazards, the hazard protection requirement and the circumstances after the hazard protection measures. I consider that these hazards are then traceable through SFRs in the BoD with a clear referencing system used in the SFRs and in other supporting documentation, including the reinforced concrete barrier substantiation reports (Refs. 57, 58 and 59). I am satisfied that the use of this system of unique reference numbers meets the intent of SAP SC.2 and is an appropriate demonstration for the purpose of GDA, noting that these schedules will need to be further developed and improved for the site-specific phase, as noted in paragraph 127. The unique identifier in the internal and external hazard schedules gives full traceability of each civil engineering safety functional requirement to the hazard schedule. This meets the intent of SAPs ECE.1 and SC.4 without ambiguity.

140. In summary, whilst further work has been identified as required for the development of the internal hazard schedules and consistency across safety case documentation, for

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[7] External hazard loads include loads associated with seismic events, meteorological conditions, site hydrology, aircraft impact and external explosion. Under meteorological conditions, loads of wind, snow, rain, ice, tornados and thermal are considered.
[8] These internal hazard loads are (outlined in Figure F-3.2-1 in Ref. 59) internal flooding, fire, explosion, dropped loads, missiles, vehicle transport impact, and high energy pipe failure. The different loads resulting from higher energy pipe failure are jet impingement, whipping pipe, overpressure, high temperature, internal flooding and blast.

civil engineering GDA, the information presented in their safety case documentation has been appropriately decoupled for the civil engineering design.

4.3.5 Inputs - Fault Studies

141. Of the faults presented in the fault schedule (Ref. 118), I have assessed those with potential to influence civil engineering design. For further, detailed assessment of postulated initiating events, fault sequences and consequences in accordance with the SAPs FA- and AV-series, see Fault Studies GDA Step 4 assessment report (Ref. 56).

142. PCSR Chapter 20 (Ref. 119) describes the inputs to civil engineering design and the relationship between other disciplines, including fault studies. PCSR Chapter 12 (Ref. 120) describes the design basis conditions and Chapter 13 (Ref. 121) describes the design basis analysis for civil engineering to be designed against. From my assessment, I found the civil engineering discipline is presented as the ‘customer’ of the design requirements, fault / hazard scenarios and derived loadings from other disciplines. I found that the design basis conditions and the design basis analysis were presented an appropriate level for the PCSR Chapters at GDA.

143. The Fault Schedule (Ref. 118) presents all the low-level safety functions (LLSFs, see paragraph 74 above) with respect to fault studies. From my assessment, I have identified the faults that have a civil engineering design implication. These relate to two low level safety functions, namely: C3-2 ‘maintain containment building structural integrity’ and C3-3 ‘maintain containment building leak tightness’.

144. Regarding the golden thread from the high level claims made in the PCSR chapters to the fault schedule, I have been able to trace the golden thread regarding confinement (C3-2 and C3-3) from PCSR Chapter 16 (Ref. 3) to the decomposition report (Ref. 19), through to the BoSC (Ref. 33), and I have identified that there is a link from Ref. 19 to the fault schedule (Ref. 118). Albeit an ambiguous link when compared to the internal and external hazard schedules, this is adequate for the purpose of GDA, as I was able to trace these two LLSFs.

145. Regarding the golden thread from the fault schedule to DSRs, I reviewed the traceability of the two civil engineering related LLSFs (C3-2 and C3-3) from the fault schedule through to the Internal Containment BoSC (Ref. 33) which identified these two design requirements from the fault schedule. The golden thread was then apparent through SFRs presented in the BoSC (Ref. 33), the associated BoD (Ref. 50) and ultimately substantiated in the DSR (Ref. 86) where I note that the DSR confirms these two requirements are captured. I found the RP’s use of the SFR schedule throughout the documents a useful way of tracing the golden thread.

146. I found the visibility of the golden thread to be adequate except for two aspects that I consider require further work as the safety case develops. These are implicitly captured in paragraph 127.

- There is not a unique identifier in the fault schedule, which means there is not a clear traceability of each civil engineering safety functional requirement to the fault schedule, as there is in the internal and external hazard schedules. This is less significant at GDA because there are only currently two low level safety functions in the fault schedule, namely C3-2 and C3-3 pertaining to confinement related to the internal containment, which were traceable. I expect an identifier in the fault schedule, similar to those present in the other schedules, to avoid ambiguity of the golden thread upon full civil engineering design, in order to meet the intent of SAPs ECE.1 and SC.4.
- The BoD for Internal Containment (Ref. 50) does not present the pressure and temperature demands on the Internal Containment from the grouped faults in the fault schedule. This means that the safety case provides no indication of

the level of inherent margin from the derived pressures and temperatures compared to the adopted design basis. This relates to the varying margin over time of the demand loads and the pressure-time and temperature-time profiles which is key in the Internal Containment. As per the intent of SAPs SC.5 and ECE.1 the safety case should present the optimism, uncertainties, and conservatism (margin). Therefore, the margins incorporated within the load input functions should be clearly visible and the derivation of the demands clearly cross referenced. This comment applies to all load inputs used by civil engineering.

147. In summary, GDA findings withstanding, I am satisfied with the traceability of the golden thread, because the design requirements are shown to originate and flow from the fault schedule (Ref. 118) in line with the intent of the SAPs SC.4 and ECE.1.

4.3.6 Hazard Combinations

148. The RP has produced appendices of the combined hazards report (Ref. 122) which state the hazard protection requirements for combined internal hazards on rooms and walls, with the associated loading. The BoD reports contain the schedule of loads and load factors applied. From my review of combinations involving internal hazards loads, I note that Ref. 122 identifies combinations that have not been considered by the civil engineering design assessment. For GDA, I am satisfied that this was due to the developing nature of the RP's internal hazard workstream as outlined above rather than an omission, see Section 4.4.2 below. For external hazards, I am satisfied that the load combinations presented in the external hazard's combination safety evaluation report (Ref. 123) are in line with the expectations of ECE.6 and EHA.6.

149. The BoD reports reference the information found in these two reports (Refs. 123 and 122), which identify the internal and external hazards load combinations for civil engineering design to address. The external hazards combination safety evaluation report (Ref. 123) specifies the hazard frequencies to be used in combinations for external hazards, which supports the BoD. Ref. 123 identifies six load combinations of external hazard loadings that may cause consequential internal hazards. Apart from the hazard combination of earthquake and high energy pipe failure (HEPF), the other internal and external hazard load combinations have not been considered in the civil engineering design. I consider this a shortcoming against RGP that is discussed further in in Section 4.4.2 of this report.

4.3.7 Beyond Design Basis & Severe Accidents

150. The RP's approach for beyond design basis and cliff edge effects of external hazards is reported in Refs. 73 and 124, and the methodologies are assessed in Sections 4.4.9, 4.6.12 and 4.10 of this report. From my assessment, I am content with the integration of this work into the safety case documentation and how it has been reported within the DSRs.

151. As part of my assessment, I note that the BoSC reports and associated SFR schedules do not explicitly specify requirements for beyond design basis conditions e.g., it is not explicit to what extent individual SFR's need to be met under beyond design basis conditions and what the associated performance criteria would be. Rather, the RP has adopted an overarching narrative informed by largely qualitative arguments. I am content with this for the purpose of GDA but emphasise that the framework for defining the detailed beyond design basis requirements will need to be established in the site-specific phase. I consider this normal business.

152. With respect to severe accident conditions, often termed design extension conditions, and the associated analysis, the two areas considered by civil engineering are the ultimate capacity of the internal containment against pressure and temperature

transients and the hazard posed from malicious aircraft impact. From my assessment of the fault analysis as it applies to civil engineering (Ref. 118), I am satisfied there is adequate traceability of the requirements for these analyses.

153. Malicious aircraft impact is treated as a beyond design basis event and the acceptance criteria are outlined in Section 4.10 of this report. The threat definitions are assessed in the ONR external hazards assessment report (Ref. 55).

154. The evaluation of the internal containment ultimate capacity is a deterministic evaluation that establishes the available margin in the design, rather than evaluating a specific severe accident scenario. The acceptance criteria are not made clear within the civil engineering safety case. I acknowledge that there is minimal guidance on what is an acceptable margin for the ultimate capacity of the internal containment. I therefore consider it RGP for the ultimate capacity to be, as a minimum, comparable to that predicted by similar designs. The RP independently reached a similar position stating in response to RQ-UKHPR1000-0855 (Ref. 6):

"The target margin is not described in IAEA SSG-53 nor European utility requirements for LWR nuclear power plants, nor other international codes and standards. According to the RGP for EPR, for the third generation of NPP, 2.5 times design pressure is recommended as the target margin for structural ultimate capacity of containment structure, this target does not include functional failure."

The RP also confirmed (in workshop #11, see Table 1 above and Ref. 10) that the EDF EPR in Finland, Olkiluoto 3 (OL3), included a requirement in the design specification for this margin not to be less than 2.5 (Ref. 10). As noted above by the RP, I am aware that this capacity in the EPR design exists for Hinkley Point C (HPC), a similar post-tensioned concrete containment design. Therefore, I am content that a minimum reserve margin in the internal containment ultimate strength of 2.5 times the design pressure represents RGP in the UK, noting that guidance on an adequate margin between design pressure and ultimate capacity is limited. I am therefore satisfied that this target margin is comparable with similar UK containment structures. For details of this ultimate capacity analysis, please refer to Section 4.6.

155. In summary, for GDA, I am content that the safety case adequately covers beyond design basis and severe accident considerations at a level appropriate for generic design. The framework for specifying beyond design basis requirements more explicitly will need development as the safety case matures, and this is implicitly captured within paragraph 127.

4.3.8 Derivation and Golden Thread of Safety Functions

156. To establish how the golden thread of safety functions input into the civil engineering design, as part of my assessment I sampled the safety function schedules for internal containment, common raft foundation and the fuel building (BFX) (Refs 33, 31, 32). I note that the SFR schedules have developed throughout GDA Step 4 alongside the design substantiation work, rather than being defined prior to (and subsequently directing) the design. I recognise that the UK HPR1000 design uses a mature design and aligning a mature design with UK context, as discussed in Section 4.3.3, this approach is to be expected. The detailed assessment for each sample area is reported in the structure-specific sections of this report, i.e., Section 4.5 for the fuel building, Section 4.6 for internal containment, Section 4.7 for common raft foundation, and Section 4.10 for the malicious aircraft impact topic. This work also informed the cross-cutting work undertaken for resolution of RO-UKHPR1000-0004 for the project-wide assessment of requirements management; see Ref. 103.

157. During my assessment, I noted the civil engineering low level safety functions align with the fundamental safety functions of IAEA SF-1 and SSR-2/1, with the four main

civil engineering functions of control of reactivity, removal of heat, confinement and shielding. The RP has also included 'extra functions' i.e., supporting functions, hazards prevention, protection and mitigation functions; see Ref. 19. These extra functions are included even when they do not necessarily directly perform the IAEA listed fundamental safety functions. The decomposition report (Ref. 19) breaks down the fundamental and 'extra' functions into higher level safety functions, and then into the low-level safety functions, F1-F6. The relevant civil engineering BOSC reports describe the low-level safety functions, and their relationship to the high-level functions is explained in Section 3 of this report. I conclude that the identification of the functions is appropriate.

158. During my assessment, I identified the civil engineering related faults in the fault schedule (Ref. 118). The fault schedule (Ref. 118) lists the fundamental safety functions for fault scenarios and events and associates a low-level safety function that is required to control or withstand the event, which are mapped to the low level safety functions, F1-F6, for civil engineering. This provides a clear and unambiguous decomposition of the safety functions within the fault schedule. The decomposition of safety functions associated with internal and external hazards does not have this detail in the schedule, instead the internal and external hazard schedules list the hazard protection measures and relevant requirements, and the BoSC reports link these to the low-level safety functions F1-F6. From my assessment, I am satisfied that the identification and the traceability of the civil engineering safety functions (from the fundamental to the high- and low-level, through to functional requirements) are in line with the intent of SAPs ECE.1 and SC.4.

159. I note that each BoSC captures the SFRs of each structure, albeit the safety functional requirements for the common raft foundation are stated in the BoSCs for each structure and are subsequently collated in the BoD for the common raft. I consider there may be an interface that has not been tested for the foundations that are outside building footprints, but for the purposes of GDA, I consider this approach is appropriate as I do not consider any information was lost compared to having a separate BoSC for the common raft.

160. As part of my assessment, I identified that each of the schedules in the BoSCs referenced a 'related safety function'. This provides a link between the low-level safety function and the detailed safety function (F1-F6). The 'upstream reference' provides a link by either referring back to the external hazard schedule (Ref. 116) or by referring to the relevant Safety Assessment report for the individual internal hazards, which are, in turn, listed in the internal hazards schedule (Ref. 117). Each safety functional requirement is given an 'engineering requirement ID', and a 'design basis' entry (DBC-1 to DBC-4) for the internal or external hazard. As stated, this provides the link from the claims in the schedules to the safety functions to be met by design. From my assessment of the sample set taken, I am satisfied that the safety functional requirements schedules contain sufficient information for the purpose of GDA to provide visibility and appropriate detail and granularity to capture the civil engineering safety functional requirements being placed on the specific parts of the civil engineering structures, systems and components. The RP has committed to ensuring that the full suite of civil engineering safety case submissions are developed for all structures to provide an appropriate, consistent level of detail, as recorded in paragraph 127.

161. The safety functional requirements schedules do not explicitly cover beyond design basis and severe accident conditions. The overall approach for beyond design basis, including cliff edge effects (Ref. 125), is assessed in Section 4.4.9 of this report, with the demonstration for the BFX and the internal containment provided in Sections 4.5.10 and 4.6.12. Related to this, I also note that malicious aircraft impact is also considered a beyond design basis hazard and this is assessed in Section 4.10. The

assessment of severe accidents associated with the internal containment is provided in Section 4.6.

162. The RP has made many commitments during the GDA assessment for future design phases. Whilst this is welcome, I note that the lists of limitations and forward commitments provided in the DSRs (Sections 9 and 13.4 of Refs. 84, 85, 86 and 88) do not appear to be systematic and there are no associated forward action point numbers. This is in contrast to other RP reports that do include a traceable system. I consider this a minor shortfall and expect that this is improved to provide traceability for the detailed and site specific design.

163. From my assessment, I reviewed the civil engineering content of PSCR Chapter 4 (Ref. 18), wherein I note the term 'safety measures' is defined as 'the technical means or measures for the purpose of achieving the functional requirements'. For civil engineering, these are often passivresifdde safety measures delivered by the civil engineering structures, and I note that in a civil engineering context within the safety case submission, the term 'safety measures' is not often used. This is included herein as a note for clarity for the reader, where the RP uses the following terminology:

- Fault schedule (Ref. 118) identifies 'safety features' e.g., 'isolation of main steam-line'.
- External hazards schedule (Ref. 116) identifies 'protection measures' e.g., 'exterior walls and roof of BRX'.
- Internal hazards schedule (Ref. 117) identifies 'protection measures' and references out to specific individual barriers in a structure.

164. In summary, I am content with the RP's derivation of the safety functions for GDA. The traceability of the links to their derivation that is documented outside the civil engineering topic area is not presented consistently within the civil engineering documents: however, I am content that the information remains traceable. The RP is aware of the need for consistency in terminology across documents and the different levels of detail provided in the safety case documentation. Furthermore, the RP has committed to ensuring the civil engineering safety case be developed to address this. I am content this area is part of the overall safety case improvements highlighted in paragraph 127 that will be tracked as normal business.

4.3.9 Classification of Structures

165. The safety categorisation of safety functions and classification of the civil engineering structures are described in paragraph 76 and Figure 5 above. For the structures sampled in this assessment, I am satisfied that the RP's categorisation and classification process for the generic UK HPR1000 civil engineering structures has been applied appropriately and is consistent with RGP for new nuclear power plants. I am therefore content that the allocated categorisation and classification shown in Figure 5, including the seismic categorisation meets the intent of SAPs ECS.1 and ECS.2.

4.3.10 Strengths of the Safety Case

166. During my assessment recorded above, I have noted the following strengths:

- The overall safety case framework for civil engineering is robust and follows a claims, arguments, evidence-based approach to the safety report structure, in line with UK context and expectations.
- The RP has presented detailed flow diagrams that clearly articulate the golden thread. The diagrams also explain how the claims arguments and evidence approach has been applied to the document structure and assists the reader in navigating efficiently through the safety case.

- The RP choosing to include the SFR schedules in the BoSC and linking back to these requirements in the DSR reports has improved the visibility of the golden thread for all intended audiences of the reports.
- The RP using a unique referencing system for the hazard schedules which can be referenced in other reports gives clear traceability of hazards and requirements across the safety case into SFRs and other documentation.
- The use of a 3D PDMS model as the single source of information for use by all disciplines and the associated design change management through TCNs and a layout modification panel has been demonstrated as appropriate for the GDA Step 4.

4.3.11 Outcomes

167. In summary, from my assessment of the RP's civil engineering safety case recorded above, I am satisfied that the overall structure, scope and limitations are appropriate for the purpose of GDA. Furthermore, I am content that the cross-cutting inputs are predominantly coherent with the exception being the improvements needed in the derivation and communication of internal hazard inputs. Within the civil engineering domain, I am satisfied with the traceability and clarity of the safety functions and the RP's use of SFR schedules.

168. From my assessment, I have raised 1 assessment finding to cover an area for improvement for the site-specific design. This is to ensure the civil engineering safety case is harmonised consistent with the latest design reference. This is detailed in Annex 4.

169. Furthermore, I have identified several minor shortfalls and normal business items in the above and subsequent sections.

4.3.12 Conclusion

170. In summary, from my assessment of the civil engineering safety case recorded above, I consider that the evidence presented is in accordance with RGP and meets the intent of the ONR SAPs. Overall, I am content that the RP has developed the safety case to a proportionate level that fulfils the purposes of GDA. I am satisfied that this provides a solid foundation from which to develop it more fully in the site-specific and detailed design phases.

4.4 Design Principles and Methods for Reinforced Concrete Primary Structures

171. The design principles and methods were assessed as far as practicable during Step 3 based on the maturity of the RP's documents. However, these methodologies have been augmented significantly during Step 4, thus requiring additional assessment. This section considers the design principles and methods that are specific to the analysis and design of reinforced concrete structures. The methods are largely generic and apply as default procedures across the SSE1 classified structures (see Figure 5, para. 76 for clarification on the classification of GDA structures). SSE2 structures are also considered: however, my assessment has focused on the differences in design methodology under seismic loading; see paragraph 30 above. The scope of this section excludes the internal containment and aircraft impact analysis, the design principles and methodologies for which are discussed in Section 4.6 and 4.10 below respectively.

4.4.1 Codes and Standards

172. The codes and standards identified by the RP are predominantly US nuclear specific design codes. The RP presents the consideration of codes and standards for each structure in the relevant BoD reports, with a site-wide consideration of codes and

standards for the generic UK HPR1000 design presented in Refs. 21 and 22. I note that these codes and standards align with other designs submitted for GDA and have widely acknowledged technical provenance. With respect to the combination and compatibility of their chosen suite of codes and standards as presented in Ref. 22, from my assessment the following points are worthy of note:

- The RP is using ACI318-08 as this is the ACI318 code version referenced by ACI349-13. However, this code version is now superseded, the latest revision being ACI318-19. I also note that during Step 4, ASCE43-05 was superseded by ASCE43-19. The use of superseded codes is generally not good practice; however, I am content that this minor shortfall is primarily an artifact of the American Concrete Institute's (ACI) and American Society of Civil Engineers (ASCE) code development programmes. I expect the latest revision of these design codes to be applied for the site-specific phase.
- In addition to the primary list of codes, the RP has referenced supplementary codes and resources. These include the ETC-C and RCC-CW, additional parts of Eurocode 2 (e.g., EN1992-1-2 and EN1992-3), MAGNOX R3 and CIRIA C766, (see Annex 2 for references). Although I consider these to be sources of RGP, their compatibility with the primary codes should not be automatically assumed; rather, it should be demonstrated. From my assessment, I note that this compatibility demonstration is not covered adequately in the RP's codes and standards report (Ref. 22). This is a minor shortfall that will require attention as the design develops.
- The RP invokes ETC-C to justify the thermal reduction factors used for the structural analysis of thermal loads and to justify the dynamic load factor for external explosion. Although the ETC-C has been superseded by RCC-CW, I note that the ETC-C 2010 Edition and UK Companion Document has previously gone through ONR review and included modifications for the UK context. Furthermore, the specific factors used by the RP remain unchanged between ETC-C and RCC-CW. Therefore, for GDA I am content with the RP's reference to ETC-C but recommend the more recent RCC-CW is referred to for the site-specific phase. I am content this is normal business.

173. From my assessment, including my review of expert advice from my TSC recorded in Ref. 28, I judge that the above minor shortfalls are not significant for the GDA process. Therefore, for GDA I am broadly satisfied that the codes and standards identified by the RP meet the intent of SAP ECS.3, (see SAP's paras 170, 171, 173 and 337), and are suitable for their intended application. However, I expect improvements to be made in this area for the site-specific phase.

4.4.2 Key Design Parameters

174. For the design life, a plant 'life span' of 100 years has been nominated in the Generic Design Parameters report (Ref. 42). This is based on a conservative envelope of 5-year construction, 60-year operation and 20-year decommissioning periods and I consider this to adequately represent the design life. This design life is clearly stated in the SFR schedules as part of the acceptance criteria for the Engineering Requirement 'D3' Durability and can be traced through to the SADR's. From my assessment, I am content that the definition and use of the design life is appropriately defined for the structures, is of sufficient duration to envelope the various stages of the facilities life and is compatible with the durability design assumptions for the structures.

175. With respect to the generic design parameters, from my assessment I am content that Refs. 42 and 43 identify and consolidate the various site wide parameters that form inputs into civil engineering. Therefore, I am satisfied that the generic design parameters feeding into civil engineering are clearly defined, consistent with other disciplines, and consistent across the project.

176. The geotechnical site parameters defined in Refs. 42 and 43 cover the following areas:

- Generic site shear wave velocity envelope definition
- Generic site allowable bearing pressure definition
- Generic site ground stiffness definition
- Compatibility of representative 'Target Site' stiffness with generic shear wave velocities
- Compatibility of generic bearing pressure presented in respective documents
- Ground water level

177. These parameters have been assessed as part of the closure of RO-0009 Action 1 (Refs. 7 and 101). From my assessment, I noted some inconsistencies with the RP's documentation: these were highlighted as residual matters 1 and 2 (Ref. 101). However, I am now satisfied that the latest revisions of the affected submissions have resolved these matters, see Refs. 26, 42 and 53. Therefore, from my assessment recorded in Ref. 101, augmented by further checks noted above, I am satisfied that the RP has justified and defined a consistent set of dynamic and static geotechnical parameters that adequately represent the GDA generic site envelope[9], as per the intent of SAPs ECE.7 and ECE.13.

178. Regarding the seismic design basis, the RP has clearly stated the seismic categorisation of the buildings identified as either 'SSE1' or 'SSE2'[10]. The seismic performance of the buildings, defined in Ref. 24, clearly states that SSE1 buildings will be designed to Limit State D and SSE2 buildings will be designed to Limit State C, both in accordance with ASCE 43-05[11]. Nevertheless, the seismic analysis methodology for all buildings regardless of their classification is similar, as described in Section 4.4.5. The main difference is the inclusion of the inelastic energy absorption factor for SSE2 structures, see Section 4.4.7. I am content that the seismic category and required performance is clearly defined for each of the structures.

179. For concrete durability, the RP has designated exposure classes in accordance with Table 4.1 of EN1992-1-1. I note that Table T-3.3-1 of Ref. 42 lists only the XS 'corrosion induced by chloride' classes. This is a simplification of the full classification, however as chemical attack (arising from contamination) is excluded from the GDA scope, I consider this to be adequate and governing[12]. The RP has declared in Ref. 42 that internal and above ground structures and internal pool structures will be specified XS1, whilst external structures below ground and the common raft will be designated XS3. I consider this classification to be appropriate given the proximity to the sea and the likely presence of airborne chlorides. I note that for more controlled internal environments this is likely to be a conservative exposure class, albeit it does accommodate the exposure that will occur during construction. This is discussed further for the Common Raft in Section 4.7.

180. With respect to the loads applied to the civil structures, these are generally specified in Ref. 42 and / or the suite of BoD reports. Further calculation of the loads is provided in the SADRs where applicable. From my assessment the loads considered within these reports are consistent with those screened in/out of the GDA (as reported in Ref. 42).

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[9] The term generic site envelope is assessed in Ref. 55 and is defined in Ref. 155 as: "To ensure that a design submitted for GDA will be suitable for construction on a variety of sites within GB, the RP should specify the 'site envelope' within which the plant is designed to operate safely. The definition of the site envelope can be as broad or narrow as the RP wishes. However, it should be unambiguous and specify any site related characteristics which have been explicitly included within or excluded from that definition".

[10] For the definitions of SSE1 and SSE2, see Figure 5, paragraph 76 above.

[11] This means that SSE1 buildings will be designed to remain essentially elastic under the design basis earthquake and SSE2 buildings will have limited permanent deformation.

[12] Although ground contamination has not been considered explicitly, the XS1 condition envelopes the AC-# / DC-# exposure classes defined in BS8500-1 for mild and medium chemical attack. The degree to which the concrete is required to resist attack from ground contamination can be addressed in the site-specific phase.

These loads are also consistent with the signposting provided in the SFR schedules and hazard schedule reports.

181. For the buildings within GDA scope, the RP has presented load schedules that provide a useful summary of the different loads applicable to the building, the structural members to which the load is applicable and the upstream reference[13]. I note that the level of detail within these load schedules is variable, with the Internal Containment load schedule representing the most detailed example. From my assessment, I have identified potential improvements to the schedules in other BoD reports. These improvements include additional references for the load definition and improved referencing between safety case documentation. The RP acknowledges the improvements required, and this requirement for further work is captured within the overall improvements noted in paragraph 127, see Section 4.3.2 above. However, I am content that the RP has demonstrated their ability to meet the intent of SAP ECE.6 to an appropriate level for the GDA phase. More specific comments are made below with respect to internal and external hazards.

182. The detailed consideration of the derivation of internal hazard loads is outside the scope of this report; this is covered by the ONR internal hazard assessment report (Ref. 41). However, from my assessment, some comments are made below on the approach adopted by the RP and how the internal hazard loads have been considered, and where necessary, rationalised for input into the civil engineering analysis and design process.

183. I note that the RP's identification of internal hazard loads for civil engineering initially only focussed on structural members where specific safety claims were made as part of the internal hazard safety case, referred to as 'Barriers'. The RP's approach to defining the loads applied to 'non-barrier' structural elements was developed to overcome the shortfalls identified in RO-UKHPR1000-0054 (Ref. 7). This process (tracked by RO-UKHPR1000-0054) was as follows:

- The RP developed a detailed methodology for the identification of internal hazard loads for the civil engineering design of non-barrier elements. The RP applied this methodology to rooms beneath the BFX Spent Fuel Pool (SFP) as a demonstration for GDA. These structures were chosen due to their function in supporting a structure that is key to nuclear safety. The RP confirmed that this methodology would be more widely applied to the rest of BFX, and other structures at the site-specific stage.
- In parallel, for the substantiation of the BFX civil structures, the RP developed a 'decoupled' set of conservative internal hazard loads for civil engineering design, based on expert judgement from the RP's internal hazard team. This was intended to improve overall understanding on the likely impact of internal hazard loads on non-barrier structural elements.
- The loads derived from the localised internal hazard analysis were subsequently checked by the RP to confirm they were bounded by the decoupled loads.

184. This decoupling methodology was implemented by the RP for the BFX facility only. Although this provides sufficient confidence for the purpose of GDA, further work remains to be done to both develop adequate internal hazard loads (see the internal hazards assessment report, Ref. 126) and substantiate the structures. This applies to the remainder of the BFX, and for non-barrier structural elements of other buildings. I am content this can be taken forward as normal business in the detailed design phase.

185. I have reviewed the methodology for processing the internal hazard loads in Refs. 57, 58 and 59. I am satisfied with the method of application and that where simplifications

[13] *For examples refer to Tables T-7-7 of Ref. 50, T-7.19-1 of Ref. 44 and T-7-2 of Ref. 53.*

are made, these are conservative and in line with RGP, predominantly, but not exclusively, ACI349-13 and EN1992-1-1.

186. Of the load combinations considered in the design, I am content that these are in accordance with ACI349 (and ACI359 for the internal containment). I note that Ref. 127 identifies several combinations of external and internal hazard loads that have not been considered in GDA, including combined earthquake and internal flooding: this is discussed further in the ONR internal hazards assessment report (Ref. 41). The RP has confirmed in Section 5.3 of Ref. 25 that, although these should be considered as design basis conditions, this load combination has not been considered in the civil engineering design at GDA. I note that hydrodynamic loads will also require consideration for this internal flooding load case. The RP claims that the governing load case of earthquake and high energy pipe failure (HEPF) is likely to bound the other combinations omitted in GDA. Based on expert advice (Ref. 28), I consider this assumption to be reasonable for the purposes of GDA. The further work required is recognised within assessment finding AF-UKHPR1000-0215.

AF-UKHPR1000-0215 – The licensee shall, as part of the site-specific design, ensure that the civil engineering design requirements include relevant combinations of external hazard and internal hazard loads.

187. The detailed consideration of the derivation of external hazard loads is outside the scope of this report, this is covered by the ONR external hazard assessment report (Ref. 55). For civil engineering, the external hazard loads are interpreted and defined in Ref. 42. From my review, two points are noteworthy.

- I note that the thermal load generated by direct solar radiation[14] is not considered at GDA. The RP has stated in Ref. 42 that this will be considered at the site-specific stage and Ref. 71 further highlights that an insulated "warm roof" system will be adopted, with the intent that this will be designed to suppress the direct solar thermal load significantly. On this basis, I consider it reasonable that this thermal load has been omitted at GDA.
- For accidental aircraft impact, the RP has conservatively considered this hazard as a design basis condition for all structures susceptible to impact. I note that the assessment of accidental aircraft impact has focussed on structural damage effects but does not appear to have considered other effects (e.g., fire/vibration/local damage). From my sample assessment of the BNX facility, the results presented in the BNX SADR (Ref. 65) suggest that the associated load combinations may govern the design of some structural elements. Therefore, if the site-specific hazard analysis indicate that accidental aircraft impact cannot be screened out (necessitating its consideration as a design basis condition), then the RP will need to give their approach further consideration. Nonetheless, for the purpose of GDA, I consider the RP's approach of including accidental aircraft impact as a design basis load condition adequate and conservative.

188. From my assessment I am satisfied with the interpretation of these generic external hazard loadings for the civil engineering analysis. I consider that RGP has been followed appropriately, for the purposes of GDA. Further evaluation of these methodologies will be necessary once site-specific hazard analysis is completed; however, I consider this normal business.

189. The load combinations and load factors are defined within each of the BoD reports, and this information is expanded upon in the SADR reports. From my assessment, I

[14] *This can generate temperatures significantly hotter than the air temperatures that are being considered as part of GDA.*

note that these combinations are generally based on ACI349, whilst the internal containment (and pertinent areas of the common raft) adopt combinations from ACI359. Additional combinations of external and internal hazards (considered non-governing) have been identified by the RP and will be considered by the civil engineering design in the site-specific phase; see paragraph 186 above. I consider that the combinations adequately cover both ultimate limit state and serviceability limit state design conditions and that those omitted are unlikely to be governing. Furthermore, I consider that the level of detail within the BoD and SADR reports meets the expectation of ONR SAP ECE.6 for the purpose of GDA.

190. With respect to partial factors, for strength design the RP has confirmed in Ref. 25 that these are typically specified in accordance with ACI349. For local effects specific to impact of internal hazard loads, ACI 349 appendix F.7 provides further guidance on ensuring the adequacy of structural elements for local effects, including penetration, perforation, scabbing, and punching shear. This is of particular relevance to internal hazard loads, where perforation and scabbing requirements have been specified for several barrier elements, to substantiate the safety case. The RP has chosen to adopt the approach outlined in Magnox R3 for quantifying the perforation and scabbing thicknesses. Although I consider this to be reasonable, I expect the design to also be compliant with ACI349. Currently, I note that the RP’s approach does not incorporate the recommendations of ACI349 clauses F.7.2.1 and F.7.2.2 that require the concrete thickness to be at least 20% greater than the minimum required to prevent perforation or scabbing. The RP has committed in their response to RQ-UKHPR1000-1632 (Ref. 6) to address this shortfall in the site-specific phase and that, if necessary, either the thickness of barriers will be increased, or concrete scabbing shields added. I am content that either option would be compliant with ACI349. This future work requirement is discussed further within the ONR internal hazards assessment report (Ref. 41) and captured within assessment finding AF-UKHPR1000-0056 therein.

191. With respect to material specification, I note that the RP has specified the principal mechanical parameters for the concrete and reinforcement steel within the BoD reports. I note that all structures adopt C40/50 strength concrete (with the internal containment adopting C50/60), with reinforcement steel specified as carbon steel grade B500C to BS4449. Details on the materials for the internal containment prestressing tendons, the internal containment liner and the spent fuel pool liner are specified and discussed within Sections 4.5 and 4.6 below respectively. The RP has not provided details on the concrete mix design for GDA; this is expected to fall outside GDA scope due to the site-dependent nature of the source materials. However, in calculating the pre-stressing losses in Ref. 81, I note that the RP is specifying silica fume for the C50/60 concrete for the internal containment[15]. The RP should be aware that there are notable features of silica fume cements (see Section 4.3.8.2 of Ref. 28) that will reduce the overall workability of this concrete and could cause constructability challenges if sections are overly congested with reinforcement. Nevertheless, I am aware that silica fume is being used for similar applications on other nuclear power plants in the UK. With appropriate mix testing, mock-ups and construction controls, the challenges arising from the characteristics of concrete mix design should not be insurmountable. Further evidence to demonstrate constructability (e.g., site trials) will be required at the site-specific stage. I consider this normal business. In summary, I am content that the material specification accords with RGP and is appropriately detailed for the purposes of GDA, in line with the expectations of SAP ECE.16 & ECE.17.

[15] *Silica fume is an established cementitious material, included in Concrete Society Technical Report 74 and referenced in relevant codes including BS8500-1. It can be used to create high strength and high durability concretes.*

4.4.3 Analysis Methodology

192. The overall methodology and how the various models are used is illustrated in Figure 7 of Annex 6. Further information is contained within the suite of method statement documents outlined in Section 3.

193. The overall structural analysis methodology adopted by the RP is outlined in Figure F-4.3-1 of Ref. 25. This figure illustrates the approach for global models. These do not represent unique models but rather a set of models for all structures sharing the common raft foundation. All three global models (ANSYS Model 1, ANSYS Model 2 and ACS SASSI Model) are based on shell-element representation of the structural elements. ANSYS Models 1 and 2 are both linear-elastic models, allowing for the linear superposition of results. The purposes of the three global models are as follows:

- ANSYS Model 1: Creation of the baseline mesh (typically 1.5m mesh size for GDA) for the development of the ACS SASSI model. Subsequently used as an intermediate step for deriving and transferring seismic member demands to ANSYS Model 2.
- ACS SASSI: Adopts the same mesh as ANSYS Model 1, with consistent nodes and elements. This is used for seismic time-history analysis, and for the determination of seismic displacements to be used in ANSYS Model 1. By sharing common meshes, displacements from the ACS SASSI model can be applied directly to ANSYS Model 1 as a load case. The ACS SASSI Model is also used for the generation of Floor Response Spectra (FRS).
- ANSYS Model 2: Refined mesh (typically 0.75m mesh size for GDA) for static load cases, and for the combination of static and seismic loads. ANSYS Model 2 outputs are used for the global structural design.

194. Following detailed expert review by the ONR TSC (Ref. 28), I am satisfied with the analysis methodology adopted by the RP for the global models and consider the use of large three-dimensional (3D) finite element analysis models to meet RGP for analysis of similar nuclear facilities.

195. This approach of using large global 3D finite element analysis models is supplemented by additional analysis models (assessed in forthcoming sections) for the following:

- Geotechnical analysis models to derive the soil spring stiffnesses that are applied to ANSYS Model 2, see Section 4.7.
- A suite of analysis models for the design for the internal containment, including a non-linear ABAQUS model for the ultimate capacity evaluation of the internal containment, see Section 4.6.
- A suite of analysis models for the assessment of aircraft impact loads, see Section 4.10.

196. Furthermore, the RP uses more detailed local models, used for either design, sensitivity analyses or verification and validation. The strategy for local models is outlined in Ref. 25, which identifies three scenario types that warranted the use of local models:

- Scenario 1 (LM-1): local models to refine the application of equipment loads.
- Scenario 2 (LM-2): local models to consider penetrations not modelled or simplified within the global model.
- Scenario 3 (LM-3): local models to consider geometrically complex configurations.

197. The RP has demonstrated a single example of each local model scenario during GDA. The examples presented are independent models, rather than local areas refined in the global model. They include a scenario from the fuel building (BFX) for LM-1, a

penetration from the internal containment for LM-2 and the internal containment gusset region for LM-3. These examples are used to inform the design, not for validation purposes and are assessed specifically in Sections 4.5 (BFX) and 4.6 (IC) below. Furthermore, additional local models and locally refined models have been considered within the model sensitivity studies, and local non-linear time history analysis models have been developed for some of the dropped load assessments.

198. Ref. 25 outlines the general methodology for creating these local models. Ref. 25 has been subject to detailed expert review by the ONR TSC (Ref. 28) and based on this I am satisfied with the RP's approach. However, I note that the implementation of local modelling is incomplete with only 3 examples provided as independent models. Furthermore, the RP has not exhaustively identified or justified the specific areas of the structure requiring local models, or the analysis strategy to deal with them in the site-specific phase. I judge this to be a minor shortfall that will require attention as the design develops. Nonetheless, although the methodologies for developing and using local models could be more detailed, I am satisfied that the RP has provided sufficient information for the purpose of GDA. Furthermore, I judge that the examples provided are an adequate demonstration of the RP's capability to meet the intent of SAP ECE.12.

199. Intrinsically linked to the analysis methodology are the software packages used; see paragraphs 81-83. From my assessment, I am satisfied that the RP is using established and reputable software packages that are widely used and accepted for analysis of nuclear facilities. I have not assessed these further and am content that verification of widely used commercial software such as ANSYS need not be as rigorous as that for lesser-known programs. Similarly, the use of linear elastic models, as used here, requires less verification than that for non-linear models. Subject to correct implementation, I am satisfied that these software packages can fulfil the intent of the ONR SAPs AV.1, AV.2 and AV.4.

4.4.4 Modelling

200. For ANSYS Model 1 and the ACS SASSI Model, Ref. 25 confirms that 4-node linear shell elements are being used, with typical elements measuring 1.5m within the mesh. Similar 4-node elements measuring 0.75m are used within the mesh for ANSYS Model 2. With respect to the mesh configuration, Section 5.1.1 of Ref. 25 refers to a document 'Guide of ANSYS Geometry Model' which it states is "used as internal technical guidance [for] specifying the modelling requirements". From my inspection of the available images of the mesh for the sampled structures, I note that most meshes are very regular, with approximately square elements that are well conditioned. However, there are instances (e.g., Figures F-9.1-6 and F-9.1-27 of Ref. 62) that suggest the RP's guide does not set limits on the aspect ratio of elements. I expect this to be validated further in the site-specific phase. This further work requirement is captured in assessment finding AF-UKHPR1000-0216.

201. With respect to boundary conditions, each of the global structural model types described in paragraph 192 (above) is surface mounted, with horizontal and vertical soil springs attached to the raft foundation nodes. Soil pressures acting on buried walls are included as loads only, which I consider is a reasonable approach for the purpose of GDA. The adequacy of the static soil springs is covered in Section 4.7. The boundary conditions applied to the ACS SASSI model are discussed in Section 4.4.5, and the boundary conditions for the suite of internal containment models are discussed in Section 4.6.

202. With respect to modelling offsets in slabs and walls, the RP's approach outlined in Ref. 25 simplifies the two-dimensional (2-D) shell element models by representing walls and slabs with a common centreline, thus ignoring offsets arising from changes in element thickness. This approach leads to moments arising from in plane forces not being

captured. Although the RP's methodology (Section 5.1.4 of Ref. 25) acknowledges the requirement to check offsets, no guidance is provided. Furthermore, Ref. 25 indicates that the offset is only being evaluated for a single scenario that is claimed to be the most serious scenario. Therefore, I am not fully satisfied that the RP has demonstrated at GDA that the modelling results derived from the simplified centreline models reliably cater for offsets in the real structure. This will need to be investigated further in the detailed design phase. This requirement for further work is captured in assessment finding AF-UKHPR1000-0216.

203. For the modelling of junctions between orthogonal elements, the RP confirmed in Ref. 25 that orthogonal elements span to common nodes at the centreline junction. Based on expert advice in Ref. 28, I am content that this is generally conservative for shear, moment and displacement arising from out-of-plane applied actions. I note that this could be non-conservative for internal actions arising from stiffness. The RP has acknowledged this in Section 5.1.7 of Ref. 25 and commits to evaluate this on a case-by-case basis. However, no evidence of this evaluation has been presented. I therefore conclude that the ANSYS models may underestimate the stiffness of spanning elements. This conclusion is focused on locations where the finite sizes of the joints (which are not modelled) have significant impact on the clear span. The further work to address this is captured in assessment finding AF-UKHPR1000-0216.

204. For openings and other local features, the RP's modelling approach outlined in Ref. 25 is to only include in the model openings with a linear dimension greater than 1.0m. I consider this a reasonable simplification for the purpose of GDA. From my assessment of the RP's process for ensuring geometrical accuracy of the models (Refs. 68 and 70), I am content that the model openings have been captured or omitted consistently in accordance with the specified approach within ANSYS Model 1 and ANSYS Model 2.

205. The RP's approach to validation and verification is provided in Refs. 68, 69 and 70. For the three global models, the validation and verification process is outlined as a series of steps, see Figure F-3.2-1 of Ref. 70. I am satisfied that this suite of checks is comprehensive and covers all major stages of the analysis process for these global models. I note that this validation process only appears to apply to the global analyses: there is no equivalent process reported for the LM-1, LM-2 and LM-3 local models, which are themselves being used for design rather than validation. I also note that improvements to the reporting could be made in the site-specific phase as the split in information between Refs. 25, 68 and 70 is cumbersome. I expect these minor shortfalls to be resolved as normal business. Nonetheless, for the purposes of GDA I am content that the RP has sufficiently met the intent of SAP ECE.15.

206. The RP carried out validation of the ANSYS Model 1 and Model 2 mesh sizes via a series of sensitivity studies reported in Ref. 68. This work resulted in two important commitments for the site-specific phase:

- For ANSYS Model 1, the 1.5m mesh is retained; however, the RP has committed to use element nodal data for design rather than results at the centre of the element. I consider this a reasonable and pragmatic approach, and the sensitivity results suggest this to be an appropriate and conservative approach for extracting loads at the end of spans.
- For ANSYS Model 2, the RP has committed to refine the 0.75m mesh size to 0.5m at the site-specific phase. I am satisfied with this commitment, given the sensitivity that is reported in Ref. 70.

207. These commitments are positive. However, I consider that the RP could improve the visibility of them in Refs. 25 and 70. Furthermore, as the mesh used for GDA is potentially non-conservative, areas of the structure where margins are limited may require localised design changes. The further work to address this is captured in assessment finding AF-UKHPR1000-0216. Nonetheless, for the purpose of GDA I am

content that the RP's investigation of mesh sensitivity accords with the intent of SAP AV.6.

208. I note that the analytical simulation of construction staging, including the detailed assessment of locked-in stresses and changes in stiffness due to concrete hydration, creep and shrinkage, is deferred by the RP to the site-specific stage. This future work required is to develop the methods, and undertake the analyses and checks, including the evaluation of creep and shrinkage load combinations. I am content that the RP's decision to omit these aspects of the analysis is consistent with my expectations for GDA and other GDA projects, and I consider this normal business for the site-specific phase.

209. In summary, my assessment has highlighted areas of further work for the site-specific phase. I judge that the conflation of these points warrants an assessment finding for this topic area, see AF-UKHPR1000-0216.

> AF-UKHPR1000-0216 – The licensee shall, as part of the site-specific design, address the following areas concerning the modelling approach using finite element analysis and the compounding effect on the design:
> - Validation of the conditioning of the finite element mesh in localised regions.
> - Demonstration of a systematic methodology for ensuring the design results that are based on simplified centreline models for the shell element of the reinforced concrete structure reliably cater for offsets in the real structure.
> - Validation to substantiate the omission of joints in the global analysis models and the potential underestimation of stiffness of spanning elements.
> - For the global analysis models refine the finite element mesh density and the post processing methodology to meet appropriate convergence criteria.

4.4.5 Seismic Analysis

210. Ref. 24 outlines the methodology for seismic analysis, with further detail provided in Refs. 60, 61 and 68. For all structures on the Nuclear Island (see Section A.1, Annex 5 below), the RP has adopted a quasi-one-step method where the SSI analysis is performed using the finite element analysis software ACS SASSI to capture the seismic response. Linear time history analysis is performed using a suite of time histories which have been matched to the target spectral shapes. The overall methodology is illustrated by Figure F-6-1 of Ref. 24. The different performance requirements associated with Limit State D (SSE1) and Limit State C (SSE2) structures are considered within the post-processing, as described in Section 8 of Ref. 24, and discussed further in paragraph 236 below. From my assessment, I consider that the methodology presented in Ref. 24 for both SSE1 and SSE2 structures is appropriate for the purpose of GDA.

211. With respect to the input motion, this has been confirmed by the ONR external hazards assessment (Ref. 55) to adequately represent the generic site envelope. Related to this, the RP has defined the operation basis earthquake (OBE) as one third of the design basis earthquake (DBE). I am satisfied that the RP's definition of the OBE follows RGP and am content that this does not require further consideration, as per NUREG 0800. The application of the input motion (control point) is at the bottom of the foundation in the ACS SASSI model, and I am satisfied that this is consistent with the target spectra definition. Overall, I am content with the definition and application of these inputs to the ACS SASSI model.

212. The generation of time histories to match these inputs is described in detail within Appendix B of Ref. 42, and broadly follows ASCE4-16. A single set of 3 orthogonal records has been used, rather than a suite of five sets as per Section 2.6.1 of ASCE4-

16. The RP has explored the effects of using a suite of five for the very soft site, by means of sensitivity studies (Ref. 68). Although I am satisfied with this for the purpose of GDA, I expect the RP to comply fully with ASCE4-16 for the site-specific phase. From my assessment, I also note that the matching of the time histories to the target spectrum is potentially over-manipulating the original records, and therefore I consider the requirement in ASCE43-19 has not been met. These points contribute to the assessment finding AF-UKHPR1000-0217.

213. From my review of the soil modelling, I consider that the use of horizontal layers over a uniform half-space is appropriate for the GDA phase. I note that the RP has only adopted the very soft soil profile (with shear wave velocity V_s of 150m/s) for the analysis of SSE1 structures on individual rafts and SSE2 structures (i.e., the non-common raft structures). This is to enable the RP to demonstrate their methodology with respect to the lower end of the generic site envelope that would correspond to the *'Target Site'* conditions. A ramification is that the seismic demands on these structures are not expected to envelope the full range of GDA soil conditions. I acknowledge the need for a targeted approach at GDA, but I note that results for the very soft soil profile suggest the soil has an isolating effect on the structural response, as demonstrated for buildings on the common raft, where seismic demands were higher for the medium soil profile. On this basis, I note that the seismic demands placed on the non-common raft foundation structures do not envelope the full range of GDA soil conditions. However, I am content that this is a minor shortfall that is not significant for the GDA demonstration.

214. For the modelling of plant and equipment, the RP has modelled these as lumped masses, with subsequent analysis of the equipment performed in a decoupled approach. The RP carried out sensitivity studies to investigate the significance of dynamic coupling effects between the supporting structures and supported equipment, particularly for major items such as the primary circuit. The results, reported in Section 4.9 of Ref. 68, indicate that the approach for the smaller items provided a reasonable approximation of the overall structural response. I note that for the sensitivity study for the reactor pressure vessel, the local response of the primary structures was affected by its inclusion, and the RP concluded that major items need to be considered on a case-by-case basis. From my assessment, I am satisfied that these approaches are appropriate for the purpose of GDA: however, it is a simplified approach and may be non-conservative. The RP will need to justify this decoupled approach on a case-by-case basis during the site-specific phase. These points contribute to the assessment finding AF-UKHPR1000-0217.

215. Regarding the input motions used for analysis of the primary circuit components, I consider that the RP has extracted the acceleration time histories at the appropriate component support locations from the SSI analyses. I note that, to account for any uncertainties in the SSI analyses, these were effectively broadened by shifting the time step by +/- 15%. The RP subsequently performed a decoupled time history analysis of these components to calculate their seismic response. For other equipment, broadened response spectra are generated from the SSI analyses to be used in the subsequent decoupled response spectrum analyses of these items. Reaction forces were extracted from these time history and response spectrum analyses, to be reapplied to the building structural models so that the local demands can be combined with the global demands for design. I consider these approaches to be appropriate for the purposes of GDA, noting that the decoupled approach requires further justification, as noted above.

216. For hydrodynamic effects[16], I am content with the RP's approach as outlined in Section 6.5.3 of Ref. 24. The RP considers the impulsive component within the SSI analysis by lumping the liquid mass to the adjacent walls and slab in the ACS SASSI model. The

[16] *This here refers to the dynamic effects arising from water in pools such as the spent fuel pond.*

convective component is calculated manually in accordance with ACI 350.3-06, and the pressure applied directly in the ANSYS Model 2, as illustrated in Figure F-7-5 of Ref. 60. From my assessment, the approach is in accordance with RGP with the exception that the RP has not considered the “Importance Factor I” in Table 4.1.1(a) of ACI 350.3-06 in the calculation of hydrodynamic forces. This is captured within the assessment finding AF-UKHPR1000-0217.

217. For GDA, the RP has evaluated the dynamic pressures acting on embedded external walls using a pseudo-static approach, which is in accordance with Section 8.2.2 of ASCE4-16. These pressures are applied directly in the ANSYS Model 2. I am content that this simplified approach is adequate for the purposes of GDA. I note that the effect of adjacent structures on the dynamic soil pressures due to SSSI has not been derived via full SSSI analysis; see paragraph 227 below. However, the lateral surcharge loads from the adjacent structure are included. Furthermore, I note that further work is needed in the site-specific phase to articulate the analysis approach for evaluating the dynamic soil pressures on embedded walls within the footprint of a structure (e.g., pre-stressing gallery) where Section 8.2.2 of ASCE4-16 applies. This is captured in assessment finding AF-UKHPR1000-0217.

218. With respect to the treatment of accidental torsion[17], the process implemented by the RP is outlined in Figure F-11-51 of Ref. 60. From my review, I consider that this approach is rigorous, and I am content that this fully complies with RGP outlined in ASCE 4-16.

219. The RP’s process for transferring the results from the seismic analysis between global models is described in Refs. 24 and 60. The results from the ACS SASSI analysis are extracted in the form of displacement time histories. These are applied to ANSYS Model 1, from which the seismic member demands are calculated and transferred into ANSYS Model 2 for development of results for structural design. The RP obtains the absolute maximum member demands by enveloping the results across all timesteps from the ANSYS Model 1 analyses; see Figure F-11-2 of Ref. 60. I consider this process to be conservative and in accordance with the intent of SAP ECE.13. In addition, floor response spectra are generated directly from the ACS SASSI analyses, and accelerations are also extracted to be used for the subsequent pseudo-static analysis of the internal containment, as described in Section 4.6 below. Overall, I am content with the approach for processing the seismic analysis results.

220. With respect to the observed structural behaviour, the RP has demonstrated that the response of the structures is heavily influenced by the soil profile assumed in the SSI analyses. The very soft soil profile resulted in the largest displacements, both in absolute terms and relative between structures. This soil condition is therefore most significant for consideration of potential interactions between closely spaced buildings, in particular the seismic joints, as discussed further in Section 4.8. The harder soil profiles resulted in higher results for accelerations, floor response spectra, member demands and base shear forces; hence softer soil profiles are less critical for these parameters. I am satisfied that the RP has provided an adequate overview of the structural behaviour under seismic loading.

221. For the generation of floor response spectra, the RP’s approach is articulated in Section 6.4.2 of Ref. 24, Section 9 of Ref. 60, and Section 9 of Ref. 61. The methodology follows the guidelines in ASCE 4-16, and as such, I am content with this approach. From my review I note that the floor response spectra have not been generated on the floor slabs; hence the vertical out of plane response is not captured. This will be non-conservative for items of equipment that are located away from the walls supporting these floor slabs. Although I do not consider this significant for GDA,

[17] *As outlined in Section 3.1 of ASCE4-16, accidental torsion is considered to address the effects of waves not propagating vertically, rotational components of ground motion, and distributions of mass and stiffness in the structure that differ from those assumed in the construction of the mathematical model.*

this over-simplification should be addressed at the site-specific phase and is captured in assessment finding AF-UKHPR1000-0217.

222. For validation and verification of the seismic methodology, this is covered by stages V3-a and V3-b of Figure F-3.2-1 in Ref. 70, and is reported in Section 3.7 of Ref. 68. The validation of the dynamic characteristics includes:

- The dynamic behaviour of the structure in ACS SASSI against the ANSYS Model 1.
- The dynamic behaviour of the soil profile using a soil column in DEEPSOIL.
- The soil-structure frequency was validated against a lumped mass model.

223. I am satisfied that these validation checks demonstrate that the dynamic behaviour of the ACS SASSI model is reasonable.

224. In addition to the sensitivity studies already mentioned (time histories – see paragraph 212, mesh size – see paragraph 206, and equipment modelling assumptions – see paragraph 214 above) the RP has investigated the sensitivity of structural damping, concrete stiffness, SSSI and embedment effects.

225. For structural damping, ASCE4-16 provides appropriate guidance and states that, for reinforced concrete with low levels of stress (Response Level 1), a damping level of 4% is appropriate. For higher stressed reinforced concrete structures, ASCE4-16 states that where significant cracking occurs (Response Level 2), a damping level of 7% is more appropriate. For GDA, the RP has adopted Response Level 2 damping (7%) no matter what stress state. The RP has carried out studies to investigate the sensitivity of adopting Response Level 1 damping for the BFX facility, with results reported in Section 4.4 of Ref. 68. The displacements and floor response spectra were shown to be relatively insensitive to the damping level adopted, particularly for the soft soil conditions where the overall response is dominated by the soil modes. For the medium soil profile, where the soil modes are less dominant, adopting Level 2 damping results in floor response spectra that can be non-conservative by up to 25% at the higher levels of the building. The element stress results were shown to be non-conservative for both the soft and medium soils conditions considered, by up to 18%. Based on these results, the RP provided a commitment to consider damping using the iterative approach in ASCE4-16 at the site-specific design stage. For the purposes of GDA, I am content with the approach taken, however the results show that this parameter will require more in-depth consideration at the site-specific stage. This requirement for further work is captured in assessment finding AF-UKHPR1000-0217.

226. If there is significant cracking in concrete elements during a seismic event, this can affect the stiffness and damping of the concrete structure, hence the design loads and floor response spectra may be affected. ASCE4-16 gives guidance on how this should be addressed in the seismic analysis of a structure and provides a proposed methodology. The proposed methodology includes analysing the structure to calculate the seismic loads, and where significant cracking is predicted by the level of stress, the analysis is repeated with a reduced stiffness for all such parts of the structure. For GDA, the RP has followed a simplified approach, assuming full uncracked stiffness properties for the concrete structures. The RP carried out a sensitivity study for the BFX facility, investigating the effect of assuming fully cracked stiffness properties for the whole structure, see Section 4.5 of Ref. 68. The sensitivity study indicated that some results are conservative, some non-conservative, and this is driven by the frequency shifts introduced by considering the reduced concrete stiffness. I am content with the RP’s approach for the purposes of GDA, however I expect the site-specific work to be in full accordance with ASCE4-16. This should include an iterative approach whereby the highly stressed elements are reanalysed with the reduced stiffness properties, as this can result in some redistribution of stresses. I capture this requirement for further work in the assessment finding AF-UKHPR1000-0217.

227. Based on consideration of the seismic weights of the buildings and guidance in Section 5.1.5 of ASCE4-16, the RP has confirmed that the facilities needing to consider SSSI are those on individual foundations adjacent to the common raft. I am content with this conclusion. The RP provided a sensitivity study investigating the SSSI effects for the BNX building (considered to represent the worst case due to its proximity to the common raft), reported in Section 4.8 of Ref. 68. The RP performed a full 3-D SASSI analysis, with and without the adjacent structures on the common raft foundation with the very soft soil (V_s of 150m/s) profile. Building responses in the form of displacements, floor response spectra and member demands were compared, all of which showed sensitivity to SSSI effects. I am content with this demonstration for GDA. I note that the RP has committed to consider SSSI at the site-specific design stage and I expect the RP to articulate the methodology in greater detail. This further work requirement is captured within assessment finding AF-UKHPR1000-0217.

228. For embedment effects, as discussed in paragraph 201 above, the baseline case for GDA assumes surface mounted conditions; hence embedment effects are not captured. To justify this decision, the RP performed a sensitivity study for embedded structures that are not located on the common raft foundation, namely the diesel generator BDB/BDV buildings. This assumed the very soft soil profile (V_s of 150m/s) and used two ACS SASSI models for both surface mounted and embedded conditions. The floor response spectra and the structural demands were compared and reported in Section 4.7 of Ref. 68. The results indicate that considering the effects of embedment are beneficial for the structural response. The RP has confirmed that embedment effects will be treated in line with ASCE4-16 for the site-specific phase. I am content with this position and the further work required is captured within assessment finding AF-UKHPR1000-0217.

229. In summary, I have identified that the seismic analysis is suitable for the soft site conditions included in the generic site envelope but includes non-conservatisms for harder sites. Furthermore, the RP's sensitivity studies have identified a number of areas of non-conservatism that should be addressed during the detailed design of the civil structures. In view of the expectations of SAP ECE.13, I consider it necessary that ONR track these issues to completion so have raised assessment finding AF-UKHPR1000-0217.

AF-UKHPR1000-0217 – The licensee shall, as part of the site-specific design, develop the GDA seismic analysis methodology to fully meet relevant good practice and address the compounding effect on the design of structures, systems and components. This should address the following aspects:

- The use of a suite of at least five sets of time histories as per Section 2.6.1 of ASCE4-16 that are selected and modified appropriately to meet the requirements of ASCE4-16 and ASCE43-19.
- Validation of the GDA assumptions for the modelling of plant and equipment to justify whether the simplified approach is conservative.
- Inclusion of an Importance Factor in accordance with ACI 350.3-06 in the calculation of hydrodynamic loads and freeboard height.
- The analysis of dynamic soil pressures on embedded walls within the footprint of a structure.
- Capture the out-of-plane response of the floor slabs in the generation of floor response spectra.
- Full compliance with ASCE4-16 to ensure the assumed level of structural damping and extent of concrete cracking under seismic loading is appropriate for the structures stress state.
- Full articulation of the structure-soil-structure interaction methodology, and analysis and full evaluation of structure-soil-structure interaction effects.
- Detailed consideration of embedment effects under site-specific conditions.

4.4.6 Thermal Analysis

230. The RP has applied ACI349 Appendix E for the analysis of thermal loads, augmented by reference to RCC-CW, EN1992-2 and fib MC2010 bulletin No. 46 (see Annex 2). This methodology is closely aligned to that in ETC-C. The approach uses a specified minimum temperature difference across a structural element (100 ºF or 56 ºC), above which thermal loads need to be considered. The approach adopted utilises simplifying assumptions that make some provision for the thermal softening, while facilitating a linear elastic analysis and standard design methods[18], which I consider appropriate for the purposes of GDA.

231. The thermal loads themselves result from Internal and External Hazards that act in combination to create bounding temperature profiles through the structural elements, derived using thermal simulations. The temperature profiles applied in the structural analysis are based on these simulations but are simplified as outlined in Section 7.6 of Ref. 25. From my assessment, I consider this to be appropriate in accordance with ACI349 Appendix E. I have not sampled the thermal simulations during GDA. However, from my assessment of the BFX and the internal containment, I am content that conservative assumptions are made, see sections 4.5.3, 4.5.6 & 4.6.3 below.

232. For the structural response, two reduction factors are being used in parallel: a stiffness factor within the modelled material definition, and a load factor with the definition of the load combinations[19]. Based on expert advice as recorded in Ref. 28, I am content with the definition and use of the stiffness reduction factors. I note that the RP has provided supplementary commentary in Appendix B of Ref. 25 that compares the factors adopted with factors calculated using ACI349 and RCC-CW. I consider that this provides a robust underpinning for the GDA methodology.

233. I note that the load reduction factors presented in Table B.7-2 of Ref. 25 are consistent with ETC-C and RCC-CW and are more conservative than equivalent factors inferred from ACI349. I am content with the values and justifications for these factors. I note that the RP's methodology applies these factors to the thermal load case results irrespective of the stress state at each element. This is potentially locally unconservative, as certain structural configurations will lead to compressive stresses that suppress cracking. I am content that the RP is aware of this and I consider resolution of this to be normal business. This is discussed further in my assessment of the internal containment gusset, see Section 4.6 below.

234. In summary, for the purposes of GDA, I am satisfied that the thermal analysis approach and justifications of the factors used meets RGP and the intent of SAPs ECE.12 and 13.

4.4.7 Design Rules

235. The methods for assessing global stability (sliding, overturning and floatation) are described in Section 6 of Ref. 26. I am satisfied with the assumptions and approaches taken, with the caveat that I note the following omissions from Refs. 24 and 26:

- Sliding is only considered at the soil-structure interface with resistance provided via friction, defined by a generic frictional coefficient of 0.6. I consider this parameter to be rather high and expect this site-specific parameter to be shown as representative of a typical waterproofing membrane in the site-specific

[18] Evaluating thermal loads accurately requires a non-linear analysis and/or design algorithm which is computationally intensive. Despite improvements in computer processing power, for large facilities such as nuclear power plants, more simplified approaches remain RGP and are appropriate for GDA.
[19] The stiffness factor represents the effects of thermal softening, the load factor represents the effect of cracking due to flexure and/or tension.

design. Furthermore, for completeness, I consider that the RP should undertake this assessment considering the impact of the drained and undrained shear strengths of the underlying soil. These points are captured in AF-UKHPR1000-0218 below.

- The methodologies do not consider concentrated forces on structural protrusions (e.g., the prestressing gallery) that will act as shear keys. In general, as highlighted in paragraphs 217 and 228, dynamic soil pressures have not been considered for embedded structures. This will need to be addressed in the site-specific design and is captured in AF-UKHPR1000-0218 below.
- The methodology for assessing the extent of uplift that is acceptable under seismic loading was articulated by the RP in relation to analysis of the BEX building in RQ-UKHPR1000-1043 (Ref. 6). The RP clarified that the ground contact ratio would be calculated in accordance with Section 3.7.2 of NUREG-0800. The RP would accept the extent of uplift if the ground contact ratio was greater or equal to 80% and if it was less, the non-linear behaviour would be assessed on a case-by-case basis. I consider this approach to be in accordance with RGP but note that this approach has not been recorded in Ref. 24.
- The methodologies for the global stability checks do not include checks for external explosion loading across all structures included in GDA scope, although I note that this check has been included in Section 10.4 of Ref. 65 for BNX.

I consider it necessary for the RP to expand their methodology in Refs 24 and 26 to resolve these points and I have captured this requirement in assessment finding AF-UKHPR1000-0218.

AF-UKHPR1000-0218 – The licensee shall, as part of the site-specific design, implement a methodology for demonstrating global stability, incorporating but not limited to the following: • Checks on the drained and undrained shear resistance of the soil and justification for the friction coefficient used to represent any waterproof membrane. • Checks against external explosion loading. • Consideration of concentrated forces on structural protrusions that will act as shear keys. • The criteria for assessing the extent of uplift that is acceptable under seismic loading.

236. For the strength design, the RP has generally adhered to the prescriptive clauses in ACI349-13 that I consider representative of RGP. From my assessment, some exceptions warrant specific attention and are discussed in the following paragraphs:

- The RP's interpretation of the ACI349-13 code rules for design of 2-D concrete elements does not adequately take account of the biaxial stress state when deriving the concrete's strength. However, in response to RQ-UKHPR1000-0805 (Ref. 6), the RP has committed for the site-specific stage to review the strength design for reinforced concrete 2-D shell elements in accordance with Annex LL of EN1992-2 which is consistent with fib MC2010 (see Annex 2). I note that the postponement of these checks into the site-specific stage does present a design risk. However, the RP's justification in RQ-UKHPR1000-0805 is that for nuclear concrete structures, concrete compression is very rarely the governing criteria, and twisting moments are relatively small compared to other resultants. I judge this qualitative justification to be reasonable for the purpose of GDA and have captured the further work requirement in the assessment finding AF-UKHPR1000-0219.

- Ref. 25 defines the criteria to evaluate when a structural element or section is classified as a discontinuous region (D-region)[20], as defined in ACI349, and defines the design checks that should be performed. The RP's workflow is illustrated in Figure F-6.3-1 of Ref. 25. I am satisfied with the methodology applied for 'simple' D-regions. When evaluating complex D-regions using local finite element models, the RP has not provided a method and / or reference(s) for determining the material design strengths that will be used. I would expect these to be based on ACI349 / ACI318 as per the 'simple' D-regions, but this is not specified in Ref. 25. I have captured the further work required in assessment finding AF-UKHPR1000-0219.
- With respect to SSE2 structures designed to Limit State C[21] , I note that the member design of SSE2 structures has not been assessed by the RP for GDA. This includes the implementation of inelastic energy absorption factors. This will need to be considered at the site-specific stage. I consider this normal business.
- For deflections, the RP is using the ACI349 approach that uses minimum element thicknesses (in terms of span to depth ratio). The RP sets limits on the maximum analytically derived deflections as fractions of the span. I am satisfied that this approach represents RGP.
- For the serviceability checks, the RP is using a combination of American and European / British codes. For crack control, the design method is based on EN1992-1-1 supplemented by EN1992-3 and the CIRIA Guide C766. Although included in the methodology, early thermal and shrinkage checks to C766 are dependent on the site-specific construction decisions regarding concrete mix and sequencing and are therefore outside the scope of the GDA. I consider this a pragmatic decision and am content that the methodology meets RGP and is adequate for GDA.

237. The above points are consolidated in assessment finding AF-UKHPR1000-0219 below.

AF-UKHPR1000-0219 – The licensee shall, as part of detailed design, resolve the following aspects associated with the strength design methodology: • The methodology for the design of 2D reinforced concrete structures within the post-processing software, is expected to take account of the biaxial stress state when deriving the concrete's strength consistent with relevant good practice. • The methodology for determining the material design strengths that will be used when evaluating complex D-regions using finite element analysis should be fully articulated consistent with relevant good practice.

238. Regarding water tightness, from my review of the RP's submissions, the provisions are shown in Table 7 below:

Table 7: Summary of the RP's submissions related to water tightness provisions

Condition	Primary line of defence	Additional lines of defence	References
External envelope – ingress protection below ground	Design of concrete elements to EN1992-3 Tightness Class 0 with no through thickness cracks.	External continuous waterproofing membrane.	Refs. 25 and 71

[20] *Examples of D-Regions are those reinforced concrete regions with high in-plane shear stresses, such as deep beams and corbels, or local design in the vicinity of anchorages and other fixings.*
[21] *For details on Limit State C and D for SSE2 and SSE1 structures respectively, see ASCE 43-05 or ASCE43-19*

Condition	Primary line of defence	Additional lines of defence	References
		Waterstops cast within concrete at all isolation joints.	
External envelope – ingress protection above ground	Design of concrete elements to EN1992-3 Tightness Class 0, with roof falls and passive gravity drainage.	External waterproofing membrane to roofs. No additional lines of defence to walls.	Refs. 25 and 71
Internal pits (unlined) – egress protection	Design of concrete elements to EN1992-3 Tightness Class 0, with no through thickness cracks.	No additional lines of defence.	BoSC and BoD reports
Internal pits (lined) – egress protection	Internal welded steel or stainless-steel liner, typically designed to strain and stress limits specified by ACI359-17	Design of concrete elements to EN1992-3 Tightness Class 0, with no through thickness cracks.	BoSC and BoD reports

239. From my assessment, whilst I am content with the RP's proposals, I note that the requirement for no through-thickness cracks to achieve the required level of water tightness could be made clearer in the reporting. Although included in the BoSC, I consider that this should be more explicitly included in the BoD reports, with the methods for verifying the absence of through-thickness cracks also made explicit in Ref. 25. Furthermore, I note that the water tightness class for the external concrete envelope is incorrectly reported in Ref. 25 as Tightness Class 1 and contradicts the BoD reports. I consider these points to represent a minor shortfall that can be resolved in future design stages, which is implicitly captured in paragraph 127.

240. For fire resistance, the RP has specified a general fire resistance period of 2 hours as the typical engineering requirement. The adequacy of this requirement is assessed by ONR in the Internal Hazards Assessment Report (Ref. 41). The RP's methodology for ensuring fire resistance is outlined in Ref. 44, where the tabulated data method in Section 5 of BS EN1992-1-2 is specified. I consider that this method is appropriate, and recognised to be conservative, but note that this is only suitable for standard fire exposure as defined by fire curves in ISO834. Non-standard fires are discussed separately for the BDB/BDV buildings, see Section 4.8.5.

241. For the seismic joints (see Annex 5, paragraph A.1.3 for locations), the RP's methodology in Section 9 of Ref. 24 is based on ASCE43-05. However, the RP acknowledges in Ref. 24 the changes in ASCE43-19 and notes that this will be considered in the site-specific phase. I am content with the methodology presented for the purpose of GDA and the commitments made and consider this normal business at the site-specific phase.

4.4.8 Design Processes

242. For the design of reinforced concrete structural elements, the RP is using an internally developed custom software programme named REINCAL. This software enables a practical design process that flows from the analysis, with relatively seamless data handling for the checks catered for within the software. This enables the RP to consider each load case independently without enveloping (enveloping only happens once reinforcement areas are calculated as the "design results"). The details and verification of REINCAL are presented in Ref. 30. This describes the formulae used by the software, cross-referencing to the relevant code clauses (mostly within ACI349 and ACI359). Ref. 30 also presents comprehensive verification calculations for salient conditions, testing the software against a standalone calculation procedure. The scope for the use of REINCAL is outlined in Ref. 25.

243. During GDA Step 4 the RP implemented software updates to include further checks against serviceability stress limits. Furthermore, the detail regarding the post processing workflow has been substantially improved in Ref. 25 via responses to RQ-UKHPR1000-0586. Due to the specialist nature of the software, this has been subject to detailed expert review during my Step 4 assessment, and this is recorded in Section 4.9 of Ref. 28. From my assessment, informed by this review, I note that the area of improvement outstanding, as highlighted in paragraph 237 above, relates to the RP's interpretation of the ACI349-13 code rules within REINCAL for the design of 2D concrete elements, which does not adequately take account of the biaxial stress state when deriving the concrete's strength. The RP's forward commitment to review this and the ensuing updates to REINCAL is captured in the assessment finding AF-UKHPR1000-0219. Notwithstanding this further work, I am satisfied that the RP's REINCAL software package has applied RGP for the post-processing of the analysis results.

244. For parts of the structure where the assumption that *'plane sections remain plane'* is not met, it is not appropriate to apply REINCAL; this pertains to D-regions. Ref. 25 clarifies that the checks for these areas will use a combination of hand calculations and local models. For the purpose of GDA, I consider this a reasonable statement, the application of which will need further assessment in the site-specific phase as part of normal business.

245. A further limitation of REINCAL is that it considers element results on an element-by-element basis, without the ability to consider an element's location within the model as is required for shear checks close to supports, or averaging results across adjacent elements. The RP recognises that these checks need to be processed outside of REINCAL, and, as part of my assessment, the key points are discussed below:

246. Out-of-plane shear resistance is enhanced in reinforced concrete close to supports, owing to the geometrical influence of the support on the angle of the governing failure plane. The adopted design codes take account of the enhancement by defining a critical perimeter at an offset from the support; shears inbound of this critical perimeter can be ignored. The RP's approach outlined in Ref. 25 stipulates that an entire finite element must lie within the critical perimeter to have its shear neglected. I consider this approach to be conservative and avoids the challenges of interpolation.

247. With respect to averaging, the RP presents the methodology in Ref. 25. For out-of-plane shear, the RP is averaging only the results of elements within four times of the element thickness range. For in-plane shear, where averaging is required along the whole section for design, the RP has committed to check the viability of the potential load path for averaging the forces. The RP highlights the example where the averaging of in-plane shear may lead to increased tension in walls that will need to be assessed. I am content that the methodologies for design checks (that are not carried out by REINCAL) are aligned with RGP, but I note that the RP has not needed to apply averaging for out-of-plane shear in GDA.

248. The RP presents the design output as a series of tables listing practical reinforcement provisions (rationalised bar sizes and spacings) that satisfy the reinforcement demands. For the purposes of GDA, I am satisfied with this level of detail. The RP has stated in Refs. 21 and 22 that anchorage and lap lengths will conservatively envelope ACI349-13 and EN1992-1-1, that detailing will be in accordance with BS8666, and that ACI349-13 Chapter 21 will be applied to earthquake resistant design. The RP has chosen not to provide full information on the associated methodologies, rules and practices that they wish to adopt, I am content with this position and note that further assessment, considered normal business, will be needed in the site-specific phase.

249. With respect to independent peer review, the RP has stated that their technical support contractor has undertaken an independent peer review of the seismic analysis and

design of the buildings, as suggested by ASCE43-19. However, it remains unclear what level of detail this independent peer review achieved and what the findings were. Nonetheless, I am satisfied with the RP's application of independent peer review during GDA and regard the lack of formal reporting as a minor shortfall that can be addressed in the site-specific phase.

4.4.9 Beyond Design Basis

250. The RP’s overarching methodologies for beyond design basis and cliff edge are reported in Refs. 73 and 124. The RP’s definition of beyond design basis refers to events that occur significantly beyond the design basis. For GDA, the RP has assessed beyond design basis explicitly for the internal containment and the hazard of malicious aircraft impact. This methodology and substantiation for these are assessed in Sections 4.6 and 4.10 below. For other structures, the qualitative arguments made under the cliff edge assessment are invoked. Therefore, this cliff-edge methodology, as presented in Figure F-3-2 of Ref. 73, is the focus of this section.

251. For internal hazards, the RP claims that as these are defined as Maximum Credible Events, and that no further evaluation is required. However, in view of paragraphs 182-186, I consider that this claim will need reviewing in the site-specific phase, once the final set of internal hazard loads are finalised. I am content that this can be considered normal business.

252. With respect to external hazards, the methodology relies upon demonstration of adequate margins and, where appropriate, some hazards being discounted if they are bounded by others. This approach of screening the external hazards from further consideration is assessed by ONR in the External Hazards Assessment Report (Ref. 55). The summary of this process is:

- For wind and tornado loading, the RP argues that the FCG3 values bound UK requirements, and can therefore be discounted from the cliff-edge evaluation.
- The evaluation of electro-magnetic interference and space weather, and heat sink specific hazards are judged by the RP to be of negligible impact to civil engineering and have not been considered further.
- Extreme water temperature and lightning are not considered further, based on these being maximum credible events. I note that for the spent fuel pool, a bounding water temperature of 100°C has been considered as a design basis thermal load.
- Flooding has been deemed site-specific and has not been reviewed as part of the civil engineering assessment.
- Aircraft crash is assessed as a “more severe beyond design basis event”, where the assessment is based in a deterministic manner based on best-estimate analysis, with different performance criteria to design basis conditions.

253. I am content with the RP’s approach and associated reasoning above. The RP assessed the hazards of low and high air temperature, snow, explosions and seismic in more detail. With respect to snow load, the RP has shown that the 10^{-5}/yr snow load is less than the vertical seismic load and the external explosion load, so the snow load can be discounted. I am satisfied with this argument.

254. For low and high air temperatures, the RP considers thermally induced stresses to be mostly self-relieving, citing ACI 349.1R-07, which provides guidance to support this claim. For the consideration of air temperature on reinforced concrete structures, I am content with this claim and argument.

255. The RP has considered a generic external explosion loading for the BFX and BEX facilities, representing typical SSE1 and SSE2 structures, respectively. The results presented in Refs. 73 and 66 show that the design basis earthquake is governing. For

the local effects, I am content with the RP's claim that, for the safety critical structures, this is bounded by the design provision against malicious aircraft impact. For the BEX, I note that local affects have not been considered as the RP has not looked at member design for SSE2 structures for GDA, see paragraph 236 above. I am content that this can be captured in the site-specific phase as part of normal business.

256. With respect to seismic loading, the RP has carried out an evaluation using 1.5DBE. For the purpose of GDA, I am content with this input and I note that this is aligned with previous GDA projects. The RP's approach has carried out the following:

- A global stability assessment for the common raft buildings and BEX.
- A seismic margin assessment of the BFX and internal containment.
- A qualitative discussion on the hierarchy of failure modes for BFX, internal containment and BEX.

257. I note that the seismic margin assessment has followed the approach to EPRI NP-6041-SL (see Annex 2). For GDA, the RP has considered the 1.5DBE load to be equivalent to the Seismic Margin Earthquake for using in the seismic margin assessment. The RP has adopted the Conservative Deterministic Failure Margin (CDFM) approach to produce a High Confidence Low Probability of Failure (HCLPF) capacity estimate for the building. I am content with this approach.

258. Related to this, I welcome the RP's commitment that reinforcement detailing will be in accordance with ACI349-13 Chapter 21. This will allow for energy dissipation due to inelastic response and is relevant for cliff edge and beyond design basis assessment. Overall, I am content with the approach the RP has adopted for evaluating seismic cliff-edge performance. The application of this to the BFX, internal containment and common raft foundation is assessed in Sections 4.5, 4.6 and 4.7, respectively.

259. In summary, the RP's overall methodology for evaluating beyond design basis and cliff edge effects relies upon bounding arguments and detailed analysis of only a subset of hazards and facilities. Nonetheless, I am satisfied that this approach is proportionate and consistent with previous GDA projects. The approach provides an adequate demonstration that SAPs EHA.18, EHA.7 can be fully met in the site-specific phase once site-specific hazard data becomes available.

4.4.10 Strengths

260. During my assessment recorded above, I have noted the following strengths:

- The RP has identified a suitable set of codes and standards for the design of the RC structures. In general, these are internationally recognised codes of practice.
- The RP has adopted standard seismic and static analysis processes using established and respected finite element codes that have widely acknowledged technical provenance.
- The RP has developed their methodologies by applying leaning and experience from previous UK GDAs.
- The RP has systematically defined the design parameters applicable to the design of RC structures.
- The RP has defined and documented a clear methodology for the design and analysis of RC structures that adheres to proven engineering practices.
- The RP has rigorously verified and validated the methods, including analysis and design tools, and design inputs and outputs, using suitably independent methods and studies.
- The RP's approach to beyond design basis and cliff-edge using the EPRI HCLPF and CDFM approaches provides confidence in the robustness of the facilities.

4.4.11 Outcomes

261. In summary, from my assessment of the RP's design principles and methodologies recorded above, I am content that these are adequately articulated, are appropriate for the purposes of GDA, and are adequately aligned with RGP and the intent of the ONR SAPs.

262. From my assessment, I have raised 5 assessment findings to address matters that require resolution as part of the site-specific or detailed design phases. As highlighted above, these are primarily associated with considering combinations of internal and external hazards, improving the overall finite element modelling approach, further validation and refinement of the seismic analysis and global stability approaches, and improving aspects of the strength design methodology. These are detailed in Annex 4.

263. Further to the above, I have identified several minor shortfalls and normal business items in the above sections.

4.4.12 Conclusion

264. Based on my assessment of the RP's design principles and methodologies recorded above, I consider that the evidence presented is in accordance with RGP and meets the intent of the ONR SAPs. These GDA methodologies developed by the RP will provide a robust foundation for the further design development necessary for site-specific design. Therefore, for this sample area, I am satisfied that the RP's demonstration has fulfilled the purposes of GDA.

4.5 Application of Design Principles and Methods – Sample 1 – BFX (SSE1 Structure on Common Raft)

265. Annex 5 (Section A.2) of this report describes the structural form of the BFX and illustrates the BFX design modifications.

4.5.1 Overview of Design Information

266. The RP has provided a full suite of safety case submissions for the BFX (i.e. including a BoSC, BoD, SADR, SAR and DSR, Refs. 31, 44, 62, 60 and 85). These are supplemented by several submissions covering specific aspects, including:

- assessment of the liner structures (Ref. 83),
- an impact assessment of changes made to the BFX GDA design that will be fully substantiated at the site-specific stage (Ref. 16),
- the constructability of specific aspects of the BFX (Ref. 78),
- a cliff edge evaluation (Ref. 73),
- the structural performance under aircraft impact (Ref. 88).

267. I note that design information is also covered in the common raft suite of documents (see Refs. 26, 76 and 77). Additionally, the RP has provided drawings, incorporated as extracts in the BoSC and BoD reports for BFX. The drawing resolution within these documents means the text (including dimensions) is not always legible. The RP has provided higher resolution drawings as separate pdf documents[22]. From my assessment, I note one minor shortfall associated with the BoSC, related to the flow diagram describing how the relevant design information links together (see Figure F-2-3 of Ref. 31). I consider that these diagrams could be improved, as they currently omit documentation related to aircraft impact and liner structures. From my review of the drawings, for the purpose of GDA, I am content that they provide an adequate level of design information regarding overall loading plans, wall and slab thicknesses, and

[22] *Of the higher resolution drawings, Refs 90, 91, 92, 93, 94, 95, 96, 97, 98 and 99 were sampled for BFX.*

assumed construction form for unique structures such as the fuel handling pits and pools, BFX roof, and fuel transfer tube.

268. The BFX has been subject to several significant design modifications during GDA Step 4. The design reference basis is outlined in Section 3.2.2 above. In summary, the main changes are as follows:

- The RP has increased the thickness for selected external structural elements (walls and roofs) forming the aircraft protection shell. This change has been implemented in the aircraft impact analyses at GDA, but not included in the detailed analysis model (ANSYS Model 2) used for the design-basis analysis. As the elements are thickened, I anticipate that results from ANSYS Model 2 will be conservative and I consider this approach reasonable for the purpose of GDA.
- The RP has increased the thickness of the common raft. This design modification has been incorporated in a version of ANSYS Model 2 but results from this model have only been processed for elements forming the common raft foundation. This approach means there are multiple variants of ANSYS Model 2 produced for the BFX. I consider this reasonable given the need to revisit the analysis at the detailed design phase and consider this normal business.

269. In parallel to these changes, several design modifications to the BFX have been introduced as a result of regulatory observations RO-UKHPR1000-0014 and RO-UKHPR1000-0056; see the Mechanical Engineering Assessment Report (Ref. 128) for details. These changes facilitate operational and maintenance activities within the BFX in a manner that meets ONR expectations. These changes have resulted in the BFX being increased in size; see Figure F-4-1 of Ref. 16 (see also Annex 5 below, paragraph A1.3.11). The RP has assessed the civil engineering impact of these BFX design modifications in Ref. 16. I assess the adequacy of this in Section 4.5.7 below.

270. In summary, I am satisfied that the suite of documentation provided for the BFX has fulfilled the scope of GDA, and I am content that the RP's management and assessment of the design modifications is adequate.

4.5.2 Structural Form and Load Paths

271. The BFX is a compartmentalised reinforced concrete structure, predominantly formed from concrete slabs for the floors, roofs and vertical shear walls. Most of the shear walls are largely continuous, except for doorways and penetrations for mechanical and electrical systems, etc. The external walls and roof of the BFX also form the aircraft protection shell for the building and are designed to withstand malicious aircraft impact loading. At the junction between the BFX and EC/BSA Zone II/BSB Zone II, the BFX shares a common vertical wall up to an elevation of +0.0m. Above this elevation, there is a seismic gap of 100mm between the BFX and the neighbouring buildings, (see Annex 5 below, paragraph A.1.3). The roof of the BFX above the fuel handling hall has a span of more than 18m. The BFX roof is unique among the GDA structures in being of composite steel-concrete construction, with steel beams and a profiled metal deck providing permanent formwork to the concrete during the construction. The RP has confirmed that the steel beams and metal deck are designed to act compositely with a first-pour (300mm thick) of the reinforced concrete slab but are subsequently ignored within calculations for the full slab in service. The constructability of this system is discussed in Section 4.5.9 below. The BFX houses numerous pool structures, including the spent fuel pool (SFP). The reinforced concrete slabs and walls forming these pools are lined with a thin steel liner attached to a framework system.

272. From my assessment, I note that the vertical load paths appear largely continuous, with transfer structures and discontinuities introduced only where driven by operational

constraints[23] . I note that, while load paths are generally continuous, there are step changes in element thicknesses for vertical walls beneath the SFP. I also note step changes in wall thickness when the external wall elements that form the aircraft protection shell become internal walls at lower elevations. Whilst I understand the rationale behind introducing these eccentricities, I note that they are generally unfavourable when considering axial forces and need to be appropriately accounted for in the design process. I discuss this further in the paragraphs below.

273. During GDA, although not formally submitted by the RP, I have seen evidence in the 3-D Plant Design Management System (PDMS) model process that the BFX layout has been derived from a holistic process considering cross-discipline requirements (see paragraph 131 above). The PDMS model includes operation and maintenance requirements for the plant. I consider that the RP's work for the regulatory observations RO-UKHPR1000-0014 and RO-UKHPR1000-0056 (Ref. 7) have further evidenced this. Overall, I consider that the structural form and load paths for the BFX appear reasonable, with evidence that buildability aspects have been considered in deciding the structural form. I am content that the load paths and structural form of the BFX have met my expectations for the purpose of GDA and will contribute to meeting aspects of SAP ECE.2.

4.5.3 Design Requirements and Parameters

<u>Safety Functional Requirements (SFRs)</u>

274. The RP has developed a detailed safety function requirements (SFR) schedule for the BFX, presented in Appendix B of the BoSC (Ref. 31). The SFR's are broken down into six structural regions: the overall structure, the structure of SFP, the pools / pits with liners (except SFP), the external walls and roofs, the internal walls and slabs, the raft foundation. From my assessment of the SFR schedules, I note the following:

- The SFR schedule clearly outlines the SSC, Design Condition, or Hazard, and provides an upstream reference for the Design Condition / Hazard. For requirements from other disciplines feeding into civil engineering, a specific reference is provided in the 'Upstream Reference' column. In some cases, the upstream referencing could be improved in the SFR schedules for the general Design Basis Conditions. I consider this a minor shortfall that can be resolved in normal business; see paragraph 127 and Ref. 103.
- The SFR schedule includes appropriate referencing to the relevant upstream schedules, and the 'Hazard protection requirement code' includes reference to the specific load case applicable to the civil engineering structure. This includes consideration of combined internal hazard loads. I note that the SFRs related to combined external hazard-internal hazard loads have not been included. The RP has committed to address these at the site-specific stage and this further work requirement is captured in assessment finding AF-UKHPR1000-0215.
- Each SFR entry is assigned a unique engineering requirement ID, which is used consistently in identifying Engineering Requirements in the Basis of Design for BFX (Ref. 44), and in the SFR Compliance schedule (Ref. 85). I am satisfied this provides a clear golden thread in line with SAP SC.4.
- The 'SSC Identification' column typically includes a cross reference to information providing specific details on the individual structural elements to which each SFR is applicable. This is particularly helpful for internal hazard loads, which may only be applicable to a room or set of rooms within the BFX.
- The RP has included additional SFR entries to clearly distinguish between the requirements for the concrete, and the requirements for the liner.

[23] *An example being the vehicle loading bay located at ground floor for transportation of spent fuel casks from the BFX that necessitates discontinuity in the vertical load path.*

- For internal hazard loads, the RP has developed additional SFR entries to capture SFRs for non-barrier structural elements.
- The RP has not developed SFR entries for the beyond design basis assessment. This is covered separately in Section 8 of Ref. 31, and is discussed further in Section 4.5.10 below. For aircraft impact, the RP presents a separate SFR schedule in Ref. 129 describing the expected performance of the BFX. The Basis of Design for BFX (Ref. 44), and the Design Substantiation Report for BFX (Ref. 85) do not make reference to SFRs related to aircraft impact. From my review of the SFR schedule in Ref. 129, I am content that the SFRs are non-contradictory and are complementary to the design-basis SFRs set out in Ref. 31, so the RP's separation of aircraft impact from the design-basis substantiation work for BFX does not appear to be problematic. The exception to this is for fire separation barriers, where, in Appendix A of Ref. 129, the SFR referred to as 'MAI-BFX-6-01' places a 3-hour fire barrier requirement on certain structural members within BFX, while Ref. 31 specifies a 2-hour fire requirement for these elements. It is not clear to me in these documents whether the RP intends to specify the bounding condition consistently across SFRs (in which case this is a reporting error), or to specify the unique requirement for each SFR and assume the designer will identify the governing requirement. The RP's response in RQ-UKHPR1000-1706 (Ref. 7) confirmed that the RP will implement the more onerous 3-hour fire barriers as specified in Ref. 129. However, I consider this should be made clear in both Refs. 31 and 129. I consider this a minor shortfall that can be resolved in normal business; see paragraph 127.

275. Further to the above, there remain typographical errors and minor inconsistencies in the individual SFR entries which require improvement. However, I am content these points are captured by the overarching safety case improvements noted in paragraph 127 and can be tracked in normal business. In summary, for the purposes of GDA, I consider the BFX SFRs to be adequately detailed, and that the SFRs and associated engineering requirements are appropriate for the range of design conditions and hazards relevant to BFX. I am satisfied the RP has sufficiently met the intent of SAP ECE.1.

<u>Loads</u>

276. The Basis of Design for BFX (Ref. 44) provides the input data for each load and clear cross-referencing to where further loading information can be found. In Ref. 62, contour plot diagrams have been provided for each load case to demonstrate how the loading information has been interpreted for the structural analysis. I consider that this information is presented clearly. From my assessment, I note the following points:

- *Live Load:* The RP has assumed a construction live load of 4kN/m^2 for GDA. Whilst I consider this is reasonable for the BFX superstructure, this could be overly restricting for the common raft foundation that may need to be trafficable (see paragraph 512 below).
- *Internal Hazard Loads:* The RP has developed internal hazard loads for both barrier and non-barrier structural elements during Step 4. Internal hazard loads for BFX are discussed in more detail in Section 4.5.5 below.
- *Crane Loads:* For the three large cranes, Section 10.4 of Ref. 62 clarifies that the static and seismic analyses have analysed seven crane location combinations considering the operating range of the cranes, and a single load case for the seismic analysis, assuming two of the cranes are at mid-span. I am content that these locations are reasonable.
- *Hydrodynamic Loads:* As noted in paragraph 216 above, I consider the RP's general method for accounting for hydrodynamic loads as appropriate for the purpose of GDA, with 50% of the fluid mass applied to the side walls to account for impulsive loading, and convective pressures applied to the ANSYS Model 2.

I note that for the final BFX load combination, the RP has summed the impulsive and convective force components. I consider this combination method conservative for calculating structural demands on the walls of the SFP as impulsive loading and convective pressures are not typically in phase. Section R4.1 of ACI350.3-06 highlights it is common practice to use the SRSS method for combining impulsive and convective force components. Some related further points of note are:

- The RP has confirmed in RQ-UKHPR1000-0771 (Ref. 6) that a more detailed hydrodynamic analysis will be undertaken during the site-specific stage. This is expected to apply a more detailed modelling approach, such as the Housner's mechanical spring mass model, thereby more correctly representing the hydrodynamic effects for the significant pools. I am satisfied that this commitment appears in line with the requirements of ASCE4-16: however, this forward commitment is not subsequently reflected in the GDA documentation. For the site-specific phase, I expect the licensee to either commit to using a more refined approach in accordance with ASCE4-16, or to provide further justification that the approach adopted at GDA is suitably bounding. The requirement for this further work is captured in assessment finding AF-UKHPR1000-0220.
- As noted in paragraph 216 above (and AF-UKHPR1000-0217), an Importance Factor24 (as per ACI350.3-06) has not been accounted for in the RP's calculations. According to the requirements of Table 4.1.1(a) ACI350.3-06, an Importance Factor of 1.50, suggested for tanks containing hazardous materials, would appear appropriate.
- For the structural design of the SFP walls and slabs, I note that the thickness requirements are driven by radiological shielding requirements. I am content with this approach, alongside the RP's arguments outlined in Section 11.5 of Ref. 60, which states that the impulsive loading and convective pressures have little influence on the design, and more reinforcement could be provided if needed.
- For the consideration of wave overtopping, the current freeboard allowance is 1.1m above the fluid surface, compared to a maximum wave oscillation of 0.83m derived using the ACI350.3-06 equations. I note that if an Importance Factor of 1.50 were considered, the freeboard allowance would exceed the factored maximum wave oscillation derived via the same method, rendering the design inadequate. This potential shortfall is acknowledged by the RP in Section 11.5 of Ref. 60, and design modifications are suggested that appear to be reasonable. Nevertheless, there remains a shortfall in the GDA SFP design with insufficient freeboard to prevent leakage of the SFP water under a DBE. This deficiency is captured in assessment finding AF-UKHPR1000-0220.

The above points are consolidated in assessment finding AF-UKHPR1000-0220 below.

AF-UKHPR1000-0220 – The licensee shall, as part of the site-specific design, resolve the following for the spent fuel pool in the fuel building: • Apply a more refined approach for the determination of hydrodynamic loads in accordance with relevant good practice or provide further justification that the approach adopted is suitably bounding. • Demonstrate that the freeboard allowance is adequate under design basis earthquake conditions in accordance with relevant good practice.

■ *Thermal Loading:* The general approach to thermal loading is discussed in Section 4.4.6: however, there are some aspects specific to BFX that I note:

[24] *The importance factor is effectively a scalar multiple on the impulsive and convective forces, and the required freeboard to prevent wave overtopping.*

- For the temperature profile of the SFP, in response to RQ-UKHPR1000-1020 (Ref. 6), the RP has stated that they have assumed the heat transfer coefficient of water on the surface of structural member is infinite. This is not stated in the documentation; however, I consider it an appropriate assumption for the analysis.
- The RP has further clarified that “Temperature in the model = Water Temperature - Reference Temperature”. This explains why the contour plot for accidental thermal loads in the SFP has an upper limit of 90°C instead of 100°C. Overall, I consider the RP’s approach for applying thermal loads to be adequate.

277. In summary, aside from the comments raised above and in Section 4.5.5, for GDA I am content that the loads considered for the BFX are appropriate and in line with SAP ECE.6.

<u>Load Combinations:</u>

278. The load combinations for the BFX are defined in the BoD (Ref. 44) and SADR (Ref. 62), with the SADR also referencing Ref. 53. I consider the information to be clear and in line with ACI349-13 that I am content represents RGP. The RP has also included a series of serviceability limit state load combinations, which are in accordance with BS EN 1990 and, thus, are compatible with design to BS EN1992-1-1. I am satisfied that the approach to these load combinations align with RGP.

279. Appendix B of Ref. 62 provides the decomposition of the load combinations into every loading permutation considered in the BFX structural analysis. From my review of this information, I note the following.

- There is clear traceability where different loading permutations are considered for a certain load type. For example, the seven different crane location combinations considered by the RP are denoted as ‘Ccr1 to Ccr7’ in load combinations 001 to 007 in Table T-10.4-7 of Ref. 62, and duplicated as required in the subsequent load combinations.
- In Section 8.1 of Ref. 62, cracking factors for normal and accidental thermal loads are identified. These cracking factors are not explicitly applied in the load combination definitions provided in Appendix B of Ref. 62, but are applied in a similar manner to the load factors applied in the different load combinations.
- Load combinations are presented for all ultimate and serviceability limit state combinations described above and are repeated for the three different soil conditions (very soft, EUR soft and EUR medium) defined for the GDA envelope.

280. Overall, except for the absence of the external hazard-internal hazard load combinations, as noted in assessment finding AF-UKHPR1000-0215 and discussed further in Section 4.5.5, I consider the load combinations considered for the BFX to be adequate and in accordance with RGP.

4.5.4 Reinforced Concrete Analysis and Design

<u>Global Analysis Model and Geometry</u>

281. The global analysis model used for the analysis and design of BFX is presented and discussed in Section 6.3.2 of Ref. 62. Therein the RP notes that all openings greater than 1.0m^2 are explicitly modelled, and that a consistent geometry is adopted across ANSYS Model 1 and ANSYS Model 2. I consider that this consistency of geometry helps eliminate errors in mapping the seismic forces from ANSYS Model 1 to ANSYS Model 2. The RP illustrates this mapping process for a sample element via a worked example in Ref. 62; this is a welcome inclusion and useful demonstration.

282. I note that in ANSYS Model 2, features such as stairs, parapets and the BFX stack are not modelled explicitly, but are instead simulated as mass elements. I consider this adequate for the static assessment. For the cranes in the fuel handling hall, the crane corbel supporting the spent fuel cask crane is modelled explicitly allowing mass elements (representing the crane) to be applied accurately at an eccentricity to the wall. For the other crane corbels, both the corbels and the cranes are simulated using mass elements, with moments applied to account for the eccentricity. I consider both methods appropriate for the purposes of GDA.

283. The validation of the geometry of the ANSYS models is provided in Refs. 68 and 70, as discussed in Section 4.4.4. Images provided in Ref. 68 show the ANSYS and PDMS models side by side and overlaid for two areas of BFX, based on design reference DR1.0 (see Figures F-3-6 and F-3-13 in Ref. 68). From my document review, these images appear to confirm that the BFX geometry in the ANSYS models suitably represents the PDMS layout configurations, capturing the major openings. I note that for all walls, any eccentricity in the centrelines is ignored, as shown in Figures F-10.3-1 and F-10.3-6 of Ref. 62. This is quite a significant simplification and is discussed further in the paragraphs below.

284. In summary, I am content that the models adopted for the BFX global analysis, albeit based on design reference DR1.0, provide an adequate demonstration for the purpose of GDA and sufficiently meet the intent of SAP ECE.12.

Global Model Results, Data Handling and Post-Processing:

285. For the global assessment of the BFX, the RP has provided a subset of analysis results for 11 different structural members. These structural members include:

- A spent fuel pool (SFP) wall and slab.
- A wall and slab beneath the SFP.
- An internal wall and slab.
- An external wall and slab.
- A non-barrier element subject to internal hazard loading.

286. The results for the SFP provide information on a key structure of interest. For the other members, I note the RP has provided little discussion describing the rationale for selecting these members. However, I observe from the results that these members do have reasonably high utilisations, so these appear to be relevant members for the RP to have focused on.

287. For each structural member, a sample set of finite elements has been chosen by the RP, for which the analysis results and reinforcement requirements are reported. The RP notes in Section 9.1 of Ref. 62 that the choice of these elements is based on engineering judgement, with the intention to identify the key horizontal and vertical sections. An example of the elements extracted for the reported SFP wall member is shown in Figure F-9.1-3 of Ref. 62. I note that the locations shown appear to coincide with the governing elements for shears and moments at the wall and slab intersections (accounting for the thickness of the wall/slab elements), as well as selecting the midspan of the wall. I consider the RP's sample set to be reasonable for the purposes of GDA.

288. For each finite element for which results are reported, Appendix E of Ref. 62 presents:

- Three governing load cases; two governing the longitudinal reinforcement in the local x- and y-directions and one for shear reinforcement.
- The required reinforcement requirement for each of these three load cases, quoted in mm^2/m for longitudinal reinforcement and in mm^2/m^2 for shear reinforcement.

- The load vector for these three load cases, which provides good traceability to independently compute the required reinforcement areas quoted.
- A comparison of the required and actual reinforcement for each finite element. The actual reinforcement is presented as bar areas, and in the form of bar diameter and pitch. I consider the latter format helpful in providing an immediate feel for how densely reinforced a particular section is.

289. The SADR for BFX (Ref. 62) does not provide a full breakdown of loads to demonstrate the enveloping process adopted by the RP. As discussed in Section 4.4.8 above, the RP has provided a comprehensive step-by-step walkthrough of the design process for ultimate and serviceability limit states, and I am satisfied with the methodology presented. The RP reports that the design is typically governed by seismic combination cases, which is consistent with the load case numbers identified in Appendix E of Ref. 62. I note that the RP has not applied any averaging of results in the outputs provided and RQ-UKHPR1000-0940 (Ref. 6) confirmed this was not required. I consider this provides confidence regarding the design margin for the sampled structural members.

290. Overall, I consider that the sample of results presented in Ref. 62 provides confidence in the outputs from the REINCAL design process and is sufficient to meet the intent of GDA. Furthermore, I am content with the RP's demonstration using quantitative methods set by the design basis codes and standards. I am content that the sample elements selected by the RP demonstrate an adequate level of performance at the ultimate and serviceability limit states.

Local Analysis and Design Methods

291. The global analysis of the BFX is augmented by local structural models for areas that have complex configurations that cannot be accurately modelled or are oversimplified in the global model. As has been noted above, the RP has not provided an exhaustive set of local models that underpin these complex areas of the BFX. For the purposes of GDA, they have provided a sample of a local structure analysis, for the detailed design of a slab area. This is denoted as model 'LM-1' by the RP, as introduced in paragraphs 196 and 197 above. The interaction between model LM-1 and the global ANSYS models used to underpin the BFX design is shown in Figure F-10.2-3 of Ref. 62. From my review, the key details of the LM-1 analysis are as follows:

- The model simulates the entire BFX, with a refined mesh size of 0.5m. Interaction with the other buildings founded on the common raft foundation is ignored, with a fixed constraint assumed at the base of the BFX.
- In Ref. 62 Appendix D, the RP notes that additional openings ≤ 1.0m diameter are modelled in the LM-1 model, and the RP appears to be following clause 13.4.2.1 of ACI349-13. Whilst I am content with the approach for modelling openings, it remains unclear which openings in this local model are not considered.
- Results are mapped onto the LM-1 model mesh based on results from the global ANSYS Model 2.
- Additional loads, including equipment reaction loads (from normal operation, accidental conditions and earthquakes) are added to the LM-1 model. I note that for equipment reactions in an earthquake, the offset of the equipment from the slab is considered. These additional loads are superimposed with those mapped from ANSYS Model 2, in accordance with the load combinations defined in the Basis of Design for BFX (Ref. 44). This is to calculate the reinforcement requirements using REINCAL. Section 10.2.2 of Ref. 62 details how the load application process demonstrates that there is no double-accounting of loads that have already been applied to either ANSYS Model 1 or ANSYS Model 2 (e.g., where lumped masses have been included for equipment).

- The element results from the LM-1 model are then processed to determine the reinforcement requirements for each load combination. These results are enveloped, and then compared with the results generated from ANSYS Model 2 for BFX. The actual reinforcement provision is based on the envelope of results from the LM-1 model and ANSYS Model 2, i.e., LM-1 is not used to disprove or refine the global model results, but only supplement them.

292. The RP has not presented the results from the LM-1 model to demonstrate the implementation of the approach. Nonetheless, I am content that the methodology is robust for the detailed design and this can be assessed as normal business in the site-specific phase[25]. In summary, I am satisfied that the methodology for this local model is rigorous and potentially conservative, meeting the intent of SAP ECE.12 and ECE.13 for the purposes of GDA.

293. With respect to the consideration of member offsets, as noted in paragraph 202 above, member offsets are generally not modelled, leading to moments arising from in plane forces not being captured. Acknowledging this, the RP has chosen a method to correct for this modelling simplification, by introducing a results post-processing step between the analysis and design. The RP has demonstrated this methodology using the wall configuration shown in Figures F-10.3-1 and F-10.3-6 of Ref. 62. The calculation presents a breakdown of the vertical load through this structural member based on different load cases (e.g., dead load, live load, seismic etc.). The moment due to eccentricity is then calculated for seven load combinations, and these moments are combined with the other forces and moments calculated for these elements from the ANSYS Model 2 results. The revised (accounting for member offsets) reinforcement requirement is then calculated for all seven load combinations, and the reinforcement requirements are enveloped to give the maximum reinforcement demand. For the example provided, the eccentricity results in an increase of up to 30% for the reinforcement demand when compared to the results taken directly from the ANSYS models, so the effect of the eccentricity is not insignificant. I consider that the method adopted is reasonable and provides some confidence into how this eccentricity can be accounted for in the design process. The RP has provided one example which is not equivalent to a systematic methodology for ensuring the design results reliably cater for element offsets. This requirement for further work is captured in assessment finding AF-UKHPR1000-0216. Additionally, I note that there will be local bursting stresses induced at changes in member thickness, and I expect these to be considered within the detailed design and reinforcement detailing as normal business in the site-specific phase.

4.5.5 Substantiation of Internal Hazard Loading

294. During Step 4, the BFX was chosen as a sample structure to assess the link between internal hazards and civil engineering. There are several internal hazard loads that must be considered for civil engineering design. For my assessment, I sampled the hazard posed from fuel assembly drop; in so doing, my review covered the following areas.

- The traceability of internal hazard loads that are inputs to civil engineering design, and how internal hazard loads and associated engineering requirements are documented within the civil engineering documentation, primarily focusing on the SFR schedules.
- The appropriateness of the SFRs for internal hazard loads.
- The adequacy and implementation of analysis methodologies for the calculation of local and global structural demands due to internal hazard loading.

[25] *At the site-specific stage, the equipment loads will also become more certain, making these additional checks more reliable.*

Traceability and Golden Thread of Internal Hazard Loads

295. As part of my assessment, I have sampled the traceability of SFRs for the fuel assembly drop case. As outlined in Figure F-3.2-1 of Ref. 59, dropped loads are one of seven different internal hazard load types that are inputs to the civil engineering design. In Ref. 130, the RP provides an overview of the different dropped load scenarios that have been identified and assessed by internal hazards. The resulting load is that which the relevant barrier is then designed to withstand (in this instance, the base slab of the SFP) to demonstrate the relevant safety function can be maintained. Appendix C of Ref. 130 provides a Hazard Schedule for dropped loads in BFX, and there are similar hazard schedules in other internal hazards safety assessment reports for all seven internal hazard load types. From my review, I note that the hazard schedules have the following features:

- Each bounding hazard is assigned a 'Hazard Reference ID'. This is an internal hazard reference to the relevant bounding load case.
- If a claim is made on a barrier element, this is identified by the 'Hazard Protection Requirement Code' column, with the barrier element identified in the 'Hazards Protection Requirement' column. This same column includes the performance requirement for the barrier.

296. In the BoSC for BFX (Ref. 31), I note the following information is included in the SFR schedule:

- An upstream reference column identifies Ref. 130 as the upstream report with the loading information.
- A 'Hazard Protection Requirement Code' column uses the same alpha-numeric reference as identified in Ref. 130. This is used as an identifier within the Reinforced Concrete Barrier Substantiation Report for BFX (Ref. 59) to provide clear linkage to the SFR schedule and the information in Ref. 130.
- A 'Hazard Protection Requirement' column, with the overarching protection requirement for the barrier.
- The SFR schedule then includes an 'Engineering Requirement ID' which contains the alpha-numeric reference adopted within the civil engineering documentation.

297. From my assessment, I consider this cross-referencing system to be clear, as it provides appropriate traceability back to the input information from the internal hazards discipline. I am content that for the purposes of GDA this framework sufficiently meets the intent of SAP SC.4 and ECE.1 and provides an adequate foundation for further enhancement in the site-specific phase.

SFRs

298. I have reviewed the SFR schedule for the BFX (see Appendix B of Ref. 31) for the dropped load case and I note the following:

- The RP has specified "Leak tightness" of the concrete as a safety function requirement, with a corresponding engineering requirement that "Structures are required to retain liquids". The RP has stated the acceptance criteria as Tightness Class 0 to BS EN1992-3 accompanied by a no through thickness crack requirement (see Table T-6-1 of Ref. 31). I am content with this requirement for leak tightness but would like to see increased visibility of this in the documentation as highlighted in paragraphs 238 and 239.
- The acceptance criteria for certain internal hazards are less specific than for other areas. As an example, for high energy pipe failure (HEPF), the SFR entry reference 'BFX-04-02-15' specifies the Engineering Requirement as: "Scabbing and perforation are not permitted unless the safety function analysis is

acceptable". In Table T-11.2-7 of Ref. 59, scabbing is predicted due to pipe whip for a number of HEPF scenarios. The RP states: "Table T-11.2-7 shows scabbing occurs for wall BFX1010VB, BFX1052VB, BFX1565VB, and related consequential analysis has been performed by internal hazards." (Ref. 59). No further information is provided in Ref. 59. The DSR for BFX (Ref. 85) presents the following the 'Substantiation Summary' column for this SFR entry: "With regard to the scabbing of some walls, corresponding safety function analysis has been performed by internal hazards in 'GHX84200047DOZJ03GN-High Energy Pipe Failures Safety Assessment Report for Fuel Building', which shows the consequence is acceptable." The RP here confirms that scabbing is predicted, and that the RP's internal hazard discipline has carried out a safety function analysis and deemed the consequence of scabbing to be acceptable.

- In response to RO-UKHPR1000-0054 (Ref. 7), the RP has presented an additional schedule of SFRs for non-barrier elements in Appendix B-1-3 in Ref. 31. I note that non-barrier elements are treated in a similar way to barrier elements, with acceptance criteria for both global and local effects. In particular, for all loads that may result in scabbing or perforation, an open statement referring to "safety function analysis" has been included (as per the quote above for SFR entry reference 'BFX-04-02-15'). However, Section 11 of Ref. 62 only presents checks for global element effects and it is unclear from the documentation whether local effects (scabbing and perforation) have also been assessed. I consider this a minor shortfall that can be resolved as normal business in the detailed design phase.
- The RP's internal hazard discipline has identified several additional combined hazard combinations (see Ref. 122), based on correlated and consequential hazards. The SFRs for these combined internal hazards are documented in Appendix B-4-2 of Ref. 31 for barrier elements. I note these internal load combinations are not considered for non-barrier elements. I consider that the RP should clarify whether combined internal hazards are applicable for non-barrier elements and provide corresponding SFRs and the design substantiation where required. I note that this matter is discussed further within the ONR internal hazards assessment report (Ref. 41) and captured within assessment finding AF-UKHPR1000-0074 therein.

299. In summary, although I have raised some areas for further development, I am content that the RP has provided an adequate framework for articulating the civil engineering SFRs for internal hazards. Further details can be added in the site-specific phase as the detailed design develops.

Methodologies

300. For the analysis of impactive loads (pipe whip, dropped loads, internal and external missiles), I note that the RP has undertaken both local and global checks on the structural elements. For the local checks, the RP has typically undertaken scabbing and perforation checks in accordance with Appendix H of MAGNOX R3 Volume 3 (see Annex 2). This approach is based on empirical formulae for calculating the required thickness of reinforced concrete elements. The formulae adopted by the RP is documented in Section 7.5.1 of Ref. 25. I am satisfied that these are appropriate for the estimation of wall thicknesses to resist scabbing and perforation. However, there are additional requirements for scabbing and perforation thickness within Appendix F of ACI349-13 that the RP is also expected to comply with. This consideration, alongside the RP's solution, are discussed in paragraph 190 above. In advance of a solution being implemented, I note there remains a shortfall in the BFX design, where for scabbing, several barriers were identified that had a factor of safety less than 1.2. As noted above, this future work requirement is discussed further within the ONR internal hazards assessment report (Ref. 41) and captured within assessment finding AF-UKHPR1000-0056 therein.

301. With respect to the input parameters, the RP has improved the clarity of the presented information in the documentation. The RP has confirmed that a missile nose shape factor of 0.84 has been assumed, corresponding to a hemispherical nose-shaped object. I consider this conservative and appropriate for a pipe whip event, as the impact is typically not normal to the impacted surface.

302. For the fuel assembly dropped load case, the RP has adopted a non-linear time history analysis approach using LS-DYNA. The analysis model is reported in Ref. 59 and illustrated in Figure F-10.3-1 of Ref. 59. From my review of this analysis, I note the following:

- The RP claims the impact velocity for the fuel assembly dropped load case is based on conservative assumptions, assuming a complete conversion of potential energy to kinetic energy in deriving the impact velocity. Moreover, the RP claims the impact mass to be conservative; see RQ-UKHPR1000-1670 (Ref. 6). I am content with these claims.
- The modelling is based on non-linear representation of steel and concrete components, using solid elements to simulate the concrete and beam elements for the reinforcement. From expert review (Ref. 28), I am content with the software, modelling and analysis used by the RP.
- The RP has developed a methodology for assessing concrete compressive strains and tensile strains in reinforcement, to assess against the requirements of IAEA SRS87. This is used to evaluate whether the design meets the safety function requirement of no through-thickness tension. I am satisfied with this approach.
- For the location of the assembly drop, the RP has committed to conduct further sensitivity studies at the site-specific stage to demonstrate that the single loading location chosen and presented in GDA is suitable bounding. The further work required for this is captured in assessment finding AF-UKHPR1000-0221.

AF-UKHPR1000-0221 – The licensee shall, as part of the detailed design of the fuel building, undertake sensitivity studies to demonstrate that the chosen spatial position for the fuel assembly drop load case is bounding in terms of in-structure demands.

- The analysis results for the fuel assembly drop are presented in Section 10.3.1 of Ref. 59. The contour plots for through-thickness tension indicate major principal strains are slightly tensile and are less than 10^{-4} for almost two thirds of the section, and less than 10^{-5} for approximately one third of the section for the governing load location. I note that, in reality, a coincidental thermal load will also be present for the fuel assembly drop scenario, which hasn't been considered by the RP and this is likely to provide some beneficial compression in the SFP slab, as demonstrated by the RP in Figure F-10.3-29 of Ref. 59. From my assessment, I judge that the magnitude of tension in the SFP slab is sufficiently low that the safety function requirement of no through-thickness tension will be met for the load case that the RP claims is governing. However, for detailed design, I expect the RP to produce a more coherent argument (SAP SC.4) and to address the related assessment finding AF-UKHPR1000-0069 recorded within the ONR internal hazards assessment report (Ref. 41).

303. For non-barrier elements, the methodology is the same as for barrier elements. This is as documented in Section 7 of Ref. 25. I am satisfied with the RP's approach. As discussed in paragraphs 183 and 184 above, the RP has used judgement to develop decoupled internal hazard loads for the analysis of non-barrier structures. The decoupled loads have been used within the global assessments. I am content with the results presented in Section 11 of Ref. 62 and concur with the suggestion the structure has adequate resistance. I note that it remains unclear whether the RP has also checked for local effects, and the full substantiation of non-barrier structural elements

against internal hazard loads will be required in the site-specific phase as the detailed design develops. The further work requirement for this is noted in paragraph 184.

304. For internal fire loads, the RP is seeking to use a fire resistance period enveloping the duration of the fire load. This enables a design approach based on BS EN1992-1-2 that I am content with; see paragraph 240 above. The RP has provided a sample of results for the BFX in Section 7 and Table T-7.2-1 of Ref. 59. I note that the RP stipulates that: “the thickness and cover of the relevant barriers against fire will be properly identified to achieve a fire resistance of two hours and ensure the barriers have adequate load bearing capacity.” From my assessment, for GDA I am satisfied with the RP’s demonstration of their method for the BFX.

305. For the analysis of load combinations identified in the BoD for BFX (Ref. 44), local and global effects must be combined. From my review of high energy pipe failure (HEPF), I note that the RP is combining elastic displacements calculated within ANSYS Model 2 for all loads except pipe whip, with the plastic displacement calculated using the MAGNOX_R3 method for the inelastic deformation due to pipe whip. The RP has carried out checks on ductility ratio and rotational capacity, in accordance with Appendix F.3 to F.5 of ACI349-13. I note that the RP has committed to adopting ductile detailing in accordance with Chapter 21 of ACI349-13, which is one of the prerequisites to adopting these ductility and rotational limits. I am satisfied with the application of this methodology for the purpose of GDA, and I observe that the utilisations are generally low for the BFX.

306. The RP has committed to considering combined external hazard and internal hazard load combinations at the site-specific stage (see paragraph 186 above, and AF-UKHPR1000-0215). For a combined internal flooding and seismic load combination, this will need to include consideration of hydrodynamic loading. I note that, for BFX, from Table T-9.1-11 of Ref. 62, the reinforcement requirements for the sampled non-barrier element have utilisation ratios up to 90%. This suggests that some structural design modifications may be necessary if seismic and hydrodynamic loads are combined with an internal flood load. I am content this can be tracked as normal business during detailed design.

4.5.6 Pool Liners

307. The design of liner structures (excluding the internal containment) is covered by the RP in Ref. 83. This is a general report, covering the method for evaluating lined structures. It contains an evaluation of the spent fuel pool (SFP) liner, as an example structure for GDA. I am content with the scope of this report and note that the SFP has unique features (i.e., importance to nuclear safety, requirements to support permanent loads in the form of spent fuel assembly racks, monitoring of leak-tightness performance) that make it more complex than other pools within the BFX. Ref. 83 links with the BoSC for BFX (Ref. 31) but is a self-contained document providing the SFRs, applicable codes and standards, load combinations, analysis methodology and assessment for the sampled SFP liner structure. My assessment has focused on the application of the RP’s methodology for the SFP liner.

308. The SFP liner and associated structural system are illustrated in Annex 5 below (Section A.2, specifically paragraph A.2.6). Individual panels are either 4mm or 6mm thick and specified as EN 1.4307 austenitic stainless steel. These are attached to a framework system, as shown in Figure F-3-2 of Ref. 83. This framework system is cast into the concrete, with the liner panels welded to this system (in situ) via full penetration butt welds. I note that the framework system has multiple purposes:

- Provides anchorage for the liner panels.
- It includes equipment anchoring plates to facilitate the attachment of various pieces of equipment for EMIT activities within the SFP.

- It is continuous along all joint (weld) lines, this therefore acts as a leak collection channel that is connected via a series of pipes to a collecting box, to allow for any leakage of the SFP to be assessed by visual inspection.

309. I am satisfied with the level of information provided for the purpose of GDA and the structural form adopted for the SFP liner. I consider this to be consistent with other SFP designs at the GDA phase.

310. The low-level safety functions (LLSFs) and SFRs applicable to the SFP liner and supporting concrete are presented in Refs. 31 and 83. From my assessment of these submissions, I note the following:

- The SFR entries on the SFP liner in Ref. 31 focus on the performance of the liner itself; meanwhile all requirements on the framework system are provided in Ref. 83. I am content with this division, but I note it does result in some unnecessary duplication in the two reports. The licensee may consider optimising this reporting in future design phases.
- Considering the construction detail of the liner, I note that the equipment anchor plate forms part of the leak tightness boundary which therefore should also fulfil Functional Requirement F4 – ‘confine radioactive material’. Despite this omission, the anchor plate is substantially thicker than the surrounding liner steel. I am content that fulfilling the leak tightness requirements will not be an onerous requirement for this component. I consider this omission a minor shortfall that can be resolved as normal business in the detailed design phase as part of the overall safety case improvements; see paragraph 127.
- SFRs are not placed on the SFP liner for dropped load scenarios. I am content that this is reasonable for an accidental drop scenario and consider this consistent with other GDA designs.
- The SFP liner is required to remain leak tight under DBE.

311. In summary, for the purpose of GDA, I am satisfied that the LLSF’s and SFR’s identified for the spent fuel pool liner are in line with the expectations of SAP ECE.1. I consider that the documenting of the SFRs and their corresponding acceptance criteria in the SFR schedules meets the intent of SAP SC.4.

312. The principal codes and standards used for the SFP liner design are:

- ACI359-17 for the assessment of the liner, including all liner welds, and for the development of load combinations,
- BS EN 1993-1-8 for the design of the framework system.

313. As the safety function of the liner is to provide containment (confinement), I am content with the use of ACI359 and consider this RGP. I consider the supporting framework system to be analogous to other steelwork structures, such as the polar crane bracket, where AISC N690-18 has been adopted. Although the RP has provided reasonable arguments for the use of BS EN 1993-1-8 (in response to RQ-UKHPR1000-1648), I note that AISC N690-18 is a nuclear-specific standard for steelwork structures, which I consider to be compatible with the load cases defined in ACI359-17. I acknowledge that BS EN 1993-1-8 has been used on other GDA projects: however, the use of this standard for the framework system design is inconsistent with the RP’s overall philosophy for codes and standards for the UK HPR1000 design. The RP’s philosophy is to adopt American codes and standards unless there are specific UK-specific requirements that are not covered by these codes. Therefore, I recommend that the RP reconsider their choice of design codes and / or provide further justification for the combined use of American and European standards. This is captured in assessment finding AF-UKHPR1000-0222.

314. The RP's analysis of the SFP liner is based on the evaluation of the following loads: self-weight (dead load), thermal and pressure loads associated with the SFP water, and seismic loading. From my review, I note that the RP's assumptions regarding dead loads and thermal and pressure loads are consistent with those assumed for the SFP concrete, as reported in Ref. 44. Within the seismic load case, I note that, for the fuel assemblies, the RP has assumed no positive connectivity between the fuel storage racks and liner. This means that any vertical load will be transferred directly to the concrete in bearing. Consistent with this, the RP has applied the lateral load from the fuel storage racks (due to friction) for the liner analysis. In reality, I consider that the primary load transfer of this lateral load will be directly into the concrete: therefore, I regard this modelling assumption to be conservative and in line with SAP ECE.13. Furthermore, the RP confirmed in response to RQ-UKHPR1000-1648 (Ref. 6) that the construction loads do not need to be considered for the SFP liner, and that the load combinations for the liner analysis are generally in accordance with ACI359 (Chapters CC-3720 and CC-3730). In summary, I am content with the loads and load combinations adopted by the RP for the SFP liner analysis.

315. For the analysis methodology, I note that the SFP liner design is substantiated through finite element analysis, with the process summarised by Figure F-12-1 of Ref. 83. Ref. 83 presents the analysis methodology, with results focussing on the finite element strain evaluation and weld and anchor bolt checks. I am content that these results are the most relevant for demonstrating the liner performance against the relevant SFRs. I consider the reporting adequate for the purpose of GDA. For the evaluation of liner strains, weld and anchor bolt checks, the liner substantiation is based on non-linear analysis of the SFP liner using ANSYS. The different models are summarised in Figure F-12-4, and an example of a finite element model is shown in Figure F-12-8 of Ref. 83. From my assessment of the analysis methodology, I note the following.

- The RP has used solid element models to capture the geometry, using 20-noded solid elements and elastic perfectly plastic material properties[26].
- The neighbouring concrete is modelled with a frictionless contact surface defined between the steel and concrete. Dimensions for the model are not provided in Ref. 83. I note that the concrete to either side of the framework system is modelled at a finite distance from this steelwork. The modelled extents will overestimate the proportion of force transferred from the liner to the concrete via the framework system. I am satisfied that this is conservative for the framework system design.
- A tied interface is used to model the welded connections. I am content that this will facilitate the extraction of contact forces to feed into weld calculations, as intended.
- Although not explicitly stated, I interpret the extent of the liner modelled to be half the width of the liner plate between adjacent framework systems. I note that the RP has adopted symmetry boundary conditions at the edge of the liner.
- To develop the bounding load cases for the analysis, the RP has assumed the governing load case is when one side of the liner plate is buckled, and the other side is not buckled. This modelling assumption would induce the maximum horizontal force through the framework system, while effectively ignoring the hydrostatic pressure acting on the liner (which in this instance would be beneficial). I am content that this is a conservative assumption.
- With regards to the RP's approach for applying seismic loads, it is unclear how the horizontal friction force from the fuel storage racks is applied. As noted in paragraph 314 above, the approach is conservatively ignoring any of the seismic lateral force being transferred through friction at the interface between the liner and concrete. Furthermore, for the load generated by concrete cracks under earthquake loading, the RP has confirmed this load case is implemented

[26] *The assumption of a perfectly plastic material model assumes that there is no work hardening and beyond the yield point the stress remains constant whilst strain increases.*

as a prescribed displacement, and it remains unclear exactly how this load is determined. I am content that this can be addressed as normal business in the site-specific phase.

316. Notwithstanding this residual ambiguity, I consider the modelling approach for the SFP liner adequate for the purpose of GDA.

317. The RP has presented results to substantiate the SFP liner and welds. From my review I note that generally the utilisations for this liner are low, with a minimum margin of 47% reported in Table T-12-1 of Ref. 83. The RP has confirmed that the SFP liner thickness of 4mm is largely based on engineering judgement (rather than the finite element results), considering welding and installation requirements. I consider this to be a pragmatic decision. The RP has evaluated the welds shown in Figure F-12-22 of Ref. 83. From my review of this evaluation, I note that the RP did not include the butt weld connecting the two segments of liner plate. Considering the shear force that must be transmitted through this weld from the liner into the framework system, I expect further justification on the performance of this weld to be presented during the detail design phase. I am content that this minor shortfall can be resolved as normal business. Nevertheless, for the purposes of GDA, I am satisfied with the results presented by the RP.

318. From my assessment of the EIMT arrangements for the SFP, I note that to facilitate the detection of leakage from the SFP, the RP has designed a network of leakage pipes that connect to the liner framework system, see Figure F-13-1 of Ref. 83. Section 13 of Ref. 83 provides further information on the filling of the SFP, with high level information on the in-service inspection and monitoring provided in Section 9 of Ref. 72. From my review, for the purpose of GDA, I am satisfied that the RP's testing, inspection and maintenance strategy for the SFP liner sufficiently meets the intent of SAPs ECE.2 and ECE.8. I note that further development will be needed at the site-specific stage to develop the design of the active monitoring systems and to evaluate and justify the pond concrete durability against any minor undetected leakage over its design life as per the intent of SAPs ECE.3, EAD.1 and EAD.2. This further work is captured in assessment finding AF-UKHPR1000-0222.

319. The above points relating to the pool liner are consolidated in assessment finding AF-UKHPR1000-0222 below.

AF-UKHPR1000-0222 – The licensee shall, as part of the detailed design of the fuel building, resolve the following for the spent fuel pool liner: • Provide justification for the mixing of design codes for assessing the design resistance of the liner framework system. • Justify the pond concrete durability against any minor undetected leakage over its design life.

4.5.7 Modifications to BFX Design at GDA

320. As highlighted in paragraph 269 above, a number of design modifications have been made to the BFX during Step 4 through the technical change note process. These were instigated by two RO's, RO-UKHPR1000-0014 and RO-UKHPR-0056 (Ref. 7). These changes were made in parallel to the main structural analysis and design of the BFX based on DR1.0 (as discussed in paragraph 66 above). The latest changes to the BFX corresponding to DR2.2 are summarised in Section 4.1 and Figure F-4-3 of Ref. 16 and also described in Annex 5 below. Due to the significance and ongoing nature of the changes to the BFX design across multiple disciplines, the RP has not repeated the civil engineering structural analysis at GDA. Rather, in response to RO-UKHPR1000-0014 and RO-UKHPR-0056 the RP presented a new report, Ref. 16, to

provide a predominantly qualitative[27] assessment of the impact of these changes on the civil engineering design. Given the ongoing nature of this design change, I am content with the RP's approach of not repeating the entire BFX analysis towards the end of GDA Step 4.

321. The RP has assessed the modifications as they impact the following aspects of the design.

- Seismic analysis of the BFX
- Structural analysis of the BFX
- Common raft analysis
- Barrier and non-barrier analysis
- Aircraft impact analysis of the BFX
- Miscellaneous.

I am satisfied with this GDA strategy, and I record my review of these areas in the following paragraphs:

322. To quantify the effects of seismic loading on the structural performance, the RP has presented a comparison of the main horizontal (in x- and y-directions) and vertical modes of vibration, based on a fixed-base model of the BFX. From my review of the results, it is evident that the revised design reference DR2.2 BFX geometry is more flexible in the vertical direction, while it is stiffer in the two horizontal directions. This change of behaviour is expected, given the changes to the geometry, and is further demonstrated by the shift in dominant frequency in the floor response spectra. The RP claims that the geometry change is likely to have a positive effect on seismic joints between adjacent buildings, due to the stiffer horizontal response of the building. I am content with this claim. As I expected, the design of the external walls remains governed by aircraft impact. In summary, I am satisfied with the seismic analysis presented for the purpose of GDA.

323. The RP has presented a qualitative assessment on the likely consequences for structural elements that are subjected to non-seismic structural loading. I consider this as a proportionate approach for the purpose of GDA. Based on the utilisations reported for sample structural elements in Section 9 of Ref. 62, I note the following:

- The sample results presented indicate high utilisations[28] for the non-barrier elements that are subjected to internal flooding (see Table T-9.1-11 of Ref. 62). I have noted this load case will also require the load combination of seismic loads including hydrodynamic effects (see AF-UKHPR1000-0215). This combination load case is likely to further increase the out-of-plane demand on these elements, resulting in modifications to wall thicknesses and / or reinforcement requirements to meet the safety function requirements.
- Sample results for structural elements that are subjected to other internal hazard loads (e.g., internal explosion, HEPF) also indicate high utilisations, and may require similar modifications.
- The increased spans are likely to result in increased serviceability limit state demands. Based on results presented in Section 9.2 of Ref. 62, the RP claims serviceability limit state demands are governing the reinforcement in several locations. I note that the serviceability limit state performance limits may dictate that further design changes are necessary for affected elements.

324. For elements that require modification, I am satisfied that the current reinforcement quantities appear reasonable and are generally amenable to increasing reinforcement bar diameters and / or additional bar layers being introduced as required. Should it be

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[27] The qualitative assessment is supplemented by simplified quantitative assessments of certain aspects of the modifications.
[28] The high utilisations are up to 0.81 in bending and 0.90 in out-of-plane shear

necessary, the RP has also suggested that wall thicknesses could be increased. I note that the licensee could decide to apply averaging of the finite element analysis results, as discussed in paragraph 247 above. In summary, the proposed changes to the BFX have resulted in increased spans for several internal walls and slabs. Based on the current utilisations, I consider it likely that changes will be required to element thicknesses / reinforcement requirements when these members are reassessed at the site-specific detailed design stage. I have raised assessment finding AF-UKHPR1000-0223 below to allow ONR to track these modifications and the associated analysis to completion.

325. For the common raft foundation, the changes made to BFX have resulted in the plan footprint of the building increasing by 4.5m in the N-S direction, and the height to width ratio of the building reducing slightly, considering the height increase of 3.1m. The RP claims this is generally beneficial for building stability and I am content with this conclusion.

326. For the aircraft impact analysis, the span sizes of the external walls have generally increased, and the design for these external elements is driven by demands due to malicious aircraft impact. The RP has presented a qualitative assessment for these members, which is assessed in Section 4.10 below.

327. In Section 4.7 of Ref. 16, the RP highlights that the increased span of the fuel handling hall may have some impact on the roof construction method. Although not clear, I consider that this increase could impact the steelwork design, impact the RP's construction strategy in terms of concrete pour sequence, and / or impact the overall thickness of the concrete roof slab. The RP will need to consider these points as the design develops in the site-specific phase, which I consider part of AF-UKHPR1000-0223 below.

328. Further modifications to the BFX have occurred between design references DR2.2 and DR3.0 as evidenced by Ref. 16. Therefore, the version that has been fully analysed at GDA is due to be superseded by these changes. However, I note that the RP has committed to revisiting the BFX analysis and design at the site-specific stage and this will require reassessment by ONR. This future work requirement is captured in assessment finding AF-UKHPR1000-0223 below. For GDA, I have taken confidence from the RP's application of the methodology in the demonstration of the substantiation of the BFX to DR1.0. I am satisfied that I have identified no obstacles to the RP repeating this substantiation for the revised BFX geometry once the geometry is confirmed.

AF-UKHPR1000-0223 – The licensee shall, as part of the detailed design, reanalyse the structure to demonstrate that the modifications to the fuel building geometry reduce civil engineering risks as low as reasonably practicable.

329. In summary, there have been significant changes to the BFX geometry during Step 4, both in terms of structural building envelope dimensions and internal room sizes. The RP's focus at GDA from a civil engineering perspective has been on demonstrating that a feasible solution can be achieved based on the modifications initiated by other disciplines. I am satisfied the RP has adequately demonstrated this for GDA and therefore from a civil engineering perspective has met the intent of RO-UKHPR1000-0014 and RO-UKHPR-0056.

4.5.8 Interfaces and Construction Details

330. Annex 5 below (paragraph A.1.3) describes the junction between the BFX and external containment/BSA Zone II/BSB Zone II, where the BFX shares a common vertical wall

up to an elevation of +0.0m. Above this elevation, there is a seismic gap of 100mm between the BFX and the neighbouring buildings.

331. From my assessment, I note that the RP's methodology for substantiating seismic joints between different buildings on the common raft foundation does not account for the critical displacement orientation which may not be in the global X- or Y-direction. For the BFX joints sampled, I am content that this shortfall appears inconsequential; however, the RP should revise their analysis methodology at the site-specific stage. I am content that this minor shortfall can be resolved as normal business.

332. Furthermore, I recommend the analysis and design of the civil engineering structure in the vicinity of the fuel transfer tube is investigated in more detail at the site-specific stage. The information provided at GDA has been limited, based on the scope of local models presented as part of GDA Step 4. I am content this will be addressed as part of normal business.

4.5.9 Constructability and Conventional Safety

333. Although construction and maintenance methodologies are developed in the site-specific design phase, the GDA considers the constructability of the design. I expect the design decisions made during the GDA phase to consider risk reduction for the whole life of the facility. Therefore, at GDA my expectation is that the RP has postulated viable construction and maintenance methodologies that underpin, inform and are consistent with the design assumptions. I expect such considerations to be cognisant of the UK legal requirements e.g., Construction (and Design) Management Regulations 2015 (CDM2015). To test this, I sampled the RP's proposed construction methodologies for the BFX roof. The BFX roof is of composite steel-concrete construction, with steel beams and profiled metal decking providing permanent formwork to the concrete during construction, while also acting compositely with a first pour of concrete forming the slab. During the operational phase, the soffit of the BFX roof is in a potentially high humidity environment as it is located above the spent fuel pool. I expect the RP to provide assurance that the design and construction assumptions facilitate and are compatible with the in-service maintenance requirements for the steelwork.

334. The RP covers this topic in two reports, Refs. 75 and 78. From my review of these reports, I note the construction methodology for the BFX roof is as follows:

- Install steel beams (the beams being sized to act as permanent falsework, providing support to the self-weight of the concrete without need for propping).
- Install metal decking (as permanent formwork to the concrete).
- Install the reinforcement to the concrete slab.
- Cast the first 300mm of concrete (ensuring not to exceed the capacity of the metal decking).
- Once the first concrete pour has reached the required strength, cast the remaining 700mm of concrete (bringing the total slab thickness to 1000mm).
- Once the second concrete pour has reached strength, apply external finishes.

335. I consider this proposed construction sequence to be logical and constructable. The RP response to RQ-UKHPR1000-1631 (Ref. 6) provided further information. From my review of this, I note the following:

- The roof steelwork involves secondary beams. These are not described by the RP and I assume these secondary beams span between, and provide restraint to, the primary beams. It is unclear whether the profiled metal deck spans onto the secondary beams (which may allow the primary beams to be placed at wider centres), or onto the primary beams.

- The profiled metal deck formwork is laid and secured with welded shear studs once the beams are in-situ (i.e., this activity will be undertaken at height).
- Workers will be reliant on a combination of personal fall-arrest harnesses and a safety net while working at height. Whilst I consider that safety nets are recognised standard practice, I note that the installation and removal of such systems typically requires access from beneath. The specific considerations of working at height are assessed by ONR (see Ref. 131).
- Although the beams are sat on corbels, the RP has not demonstrated that they have made consideration in the corbel design for improved access to the connections.
- The reinforcement to the first 300mm of concrete is placed first, including the main through-thickness reinforcement, which projects upwards out of the initial 300mm layer of concrete. The mitigation of the associated conventional safety risk to operatives, and the inconvenience posed by these bars is not clearly articulated in the construction methodology proposals.
- Although not detailed, the RP indicates that temporary structures may be erected internally, possibly using the crane and/or cast-in support points, to provide access for through-life maintenance. The methodology around these structures and the temporary load paths are not clearly articulated in the proposals.

336. Collectively, the evidence presented by the RP indicates that the design assumptions are consistent with a postulated construction sequence. The evidence also suggests that the RP is not accustomed to considering constructability and conventional health and safety for the UK context (e.g., CDM2015). In this situation, I would expect the design to be more construction and maintenance led, and this may result in the structures needing to be reanalysed or redesigned to accommodate certain aspects of the construction or EIMT. However, I am content that this can be improved upon as normal business in the site-specific phase.

4.5.10 Beyond Design Basis / Cliff-Edge Evaluation

337. For the BFX, the RP has presented a seismic margin assessment in accordance with the expected guidance as stated in EPRI NP-6041-SL (see Annex 2). Furthermore, the RP has presented a qualitative discussion on the hierarchy of failure modes for BFX to give an indication of the beyond design basis response. The global stability methodologies (assessed in paragraph 235 above), have been applied to the common raft facilities which are assessed in Section 4.7.4 below. For the purposes of GDA, I am content that this is an appropriate level of demonstration that accords with the expectations of SAPs ECE.1, EHA. 7 and EHA.18.

338. For the seismic margin assessment, the RP has adopted the CDFM approach to produce a HCLPF capacity estimate for the building. The HCLPF output is an acceleration value. For GDA, the RP compares this acceleration (measured in gravitational acceleration, ‘g’) with the 1.5DBE zero period acceleration of 0.45g. If the HCLPF value is greater than 0.45g, the performance is considered acceptable[29]. I consider this is in accordance with guidance in Table 6-1 of EPRI NP-6041-SL. For the BFX, the RP states that the predominantly lateral load path is through in-plane shear behaviour of the main vertical walls. On this basis, the seismic margin assessment is undertaken for structures except the internal containment, considering the performance of the walls, in accordance with the empirical equations as set out in EPRI NP-6041-SL. The forces considered for this assessment are extracted from ANSYS Model 2 for seismic and non-seismic loads. The RP applied these seismic loads to the soft soil condition only.

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[29] For structures except the internal containment, the RP considers the following load combination: 1.0D +1.0L +1.0 E_{SME}, where E_{SME} is equivalent to 1.5DBE .

339. In Ref. 73, the RP originally presented calculations of the HCLPF capacity, based on what they considered to be the dominant walls for lateral load transfer in the east-west and north-south direction. These walls are highlighted in yellow in the top image of Figure F-4-3 of Ref. 73. I note that in Ref. 73, the RP does not explicitly state whether other walls in BFX may be more critical for considering cliff-edge effects. Other walls may be more critical if there are comparable shear stress demands but with very different aspect ratios and large openings. To provide confidence on the HCLPF capacity at a building level, the RP has summed the HCLPF capacity of the main shear walls and compared this with the total seismic base shear under 1.5DBE. I note that this is a simplified assessment. Based on these calculations, the RP revised the number of walls considered in calculating the total HCLPF capacity in the north-south direction, as shown in the bottom image of Figure F-4-3 of Ref. 73. The RP reports the results from the HCLPF assessment for each individual wall in Table T-4-4 of Ref. 73, with a critical HCLPF value of 0.78g, that equates to a margin of 1.73 on the target peak ground acceleration of 0.45g. I am content that the structural performance is deemed acceptable regarding the HCLPF capacity calculations.

340. The RP has also considered the medium soil condition for a sample wall, reported in Table T-4-7 of Ref. 73. From my review, I note that the HCLPF value reduced from 1.09g to 0.84g, a reduction of 23%. Considering this reduction of 23% as a crude reduction applied to the other main shear walls, I am content that the HCLPF value should still be above the target peak ground acceleration of 0.45g.

341. With respect to the spent fuel pool, the RP presented the HCLPF capacity for the walls beneath the spent fuel pool for soft soil conditions, noting a critical capacity of 2.25g. I consider the calculations presented by the RP provide confidence on the cliff-edge performance of the seismic load path beneath the spent fuel pool.

342. In Section 4.2.2.3 of Ref. 73, the RP has recognised that factors (such as soil conditions, wall aspect ratios and significant openings) may impact the results of the cliff-edge evaluation. The RP has committed to considering these factors in the cliff-edge evaluation at the site-specific stage. I am satisfied with this commitment to develop the methodology for the site-specific phase. I expect the RP to consider the load transfer to these main shear walls more thoroughly in detailed design. I am content this can be progressed as normal business.

343. In summary, I am satisfied that the cliff edge evaluation for the BFX under seismic loading demonstrates large margins. This indicates not only an absence of cliff edge effects, but also adequate beyond design basis capability.

344. With respect to the qualitative discussion on hierarchy of failure modes, this is presented in Section 4.2.3 of Ref. 73. The RP presents several arguments for inherent conservatisms in the design. A number of these are due to engineering practice and compliance with codes. I note that the claimed conservatism of the EUR spectrum, although expected, cannot be quantified fully until the site-specific stage. Furthermore, the RP states that the seismic detailing is in accordance with Chapter 21 of ACI349-13. I am satisfied that this provides confidence in the performance of the structure under earthquake loading, with a ductile response for certain failure modes. The RP acknowledges that the failure of the shear walls may be brittle and argues that the margin is adequate to maintain required structural performance. Based on the results from the seismic margin assessment, and the commitment to detailing in accordance with Chapter 21 of ACI349-13, I consider the RP's claim and argument to be reasonable.

345. The RP acknowledges the current low factor of safety against sliding for the common raft foundation (see AF-UKHPR1000-0218). The RP proceeds with the evaluation on the basis suggesting that there are options to implement either modifications to the analysis methodology, and / or implement design changes that result in sliding no

longer being a critical failure mode. I consider this a pragmatic approach to the cliff-edge evaluation.

346. For the out-of-plane performance of the spent fuel pool walls, the RP notes that the contribution of seismic loading was relatively small when compared to other actions, in particular thermal loading. In addition, considering the adoption of best-estimate properties and removing strength reduction factors, the RP calculated a margin of 2.33DBE before yielding was expected. The RP also presented results for liner strains, which demonstrate considerable margin, even if considering a 1.5DBE load case. A lower margin of 1.39DBE was reported for the spent fuel pool slab based on a similar approach. The RP argues this is the margin to first yield, and the analysis does not consider any additional damping due to cracking / inelastic behaviour. I recognise that the liner is specified as stainless steel and has a high level of ductility. I am content with the RP's claims and arguments regarding the results representing the margin to first yield, as I expect any failure of the reinforced concrete to be a flexural failure, and therefore ductile, in this case.

347. In summary, for the purposes of GDA, I consider the RP's arguments and level of assessment to be adequate to meet the intent of SAP ECE.1 (specifically SAP paragraph 334) and the overarching principles of EKP.1 and EKP.3. The results indicate that the structure generally appears to have adequate margin for in-plane shear demands. The RP's commitment to use seismic detailing in accordance with Chapter 21 of ACI349-13 provides confidence that for these (and other) failure modes, the structure will be able to respond in a ductile manner. For the site-specific phase, I expect this area to be evaluated in greater detail, particularly the spent fuel pool slab. I consider this to be normal business.

4.5.11 Strengths

348. During my assessment recorded above I have noted the following strengths:

- The global 3D finite element models for the static and dynamic analysis and associated verification, validation and sensitivity analyses accord with RGP for nuclear power plants.
- The SFP follows RGP in terms of its design and the inclusion of leak detection systems.
- The results from the beyond design basis and cliff-edge evaluation using the EPRI HCLPF and CDFM approaches, alongside qualitative discussion of the failure modes, provides confidence in the robustness of the BFX.

4.5.12 Outcomes

349. In summary, from my assessment of the BFX recorded above, I am content that the RP has presented a thorough demonstration their design principles and methodologies. I am satisfied that the RP's design basis analysis for the BFX adequately meets RGP and the intent of SAP ECE.13. Furthermore, I am satisfied that the RPs beyond design basis evaluation indicates that adequate margins are available to satisfy SAPs ECE.1 and ECE.2 (specifically SAPs paragraphs 334 and 337).

350. From my assessment, I have raised 4 assessment findings to address matters that require resolution as part of the site-specific or detailed design phases. As highlighted above, these are primarily associated with refining the hydrodynamic loads and justifying the freeboard allowance for the SFP, ensuring the SFP dropped load case is bounding, substantiating the approach for the SFP liner, and fully reanalysing the revised BFX building geometry. These are detailed in Annex 4.

351. Further to the above, I have identified several minor shortfalls and normal business items in the above sections.

4.5.13 Conclusion

352. Based on my assessment of the BFX above, for GDA I am content with the RP's demonstration of their design principles and methodologies. I consider that the evidence presented is in accordance with RGP and meets the intent of the ONR SAPs. Therefore, for the BFX sample area, I am satisfied that the RP's demonstration has fulfilled the purposes of GDA.

4.6 Application of Design Principles and Methods – Sample 2 – Internal Containment

353. The information presented in Annex 5 of this report (Section A.3) describes the structural form of the internal containment.

4.6.1 Design Requirements and Parameters

354. The SFRs relating to the BRX are presented in Appendix B of the BRX BoSC (Ref. 32). This covers the safety functions and engineering requirements for the internal containment, external containment, passive water tank, and the portion of the common raft under the BRX. For the internal containment liner, the engineering requirements are articulated in Appendix A of Ref. 82 and these link back to the SFR schedule in Appendix B of Ref. 32. From my assessment, I note the following minor shortfalls:

- The RP has included internal fire in the SFR schedule for the internal containment (Appendix B of Ref. 32), however the engineering requirements breakdown in Appendix A of Ref. 82 does not include this. I note that for GDA no additional analyses is presented to demonstrate this requirement on the basis of the stud performance and failure hierarchy, as discussed in Section 4.6.8.
- There is a lack of clarity between the presented information with respect to the SFR for leak tightness of the internal containment and the design criteria for the various constituent SSCs. The RP has confirmed in RQ-UKHPR1000-0604 (Ref. 6) that leak-tightness testing will be undertaken to demonstrate the global leak-rate is less than 0.3%. However, the RP has not clarified the extent of the local testing required to demonstrate this, although I am content that this consideration will have little influence on the civil engineering design.

In summary, the safety case for the internal containment has improved significantly throughout GDA and, from my review of the internal containment, I am satisfied for GDA with the completeness and level of detail the RP has achieved. Moving forward into the site-specific phase, I judge that the framework established by the RP during GDA will provide a robust basis upon which the intent of SAPs SC.4 and ECE.1 can be fully met.

355. The codes and standards used for the design of the post-tensioned concrete in the internal containment and of its liner (and its components) are listed in Section 5 of the BoD report (Ref. 50) and Section 4 of Ref. 82. From my assessment, I note the following:

- The principal code adopted by the RP is ACI 359-17. This is augmented with European codes for concrete design, including serviceability, fire and reinforcement detailing, in line with UK requirements. Furthermore, the RP has referenced the European nuclear codes ETC-C and RCC-CW to justify the thermal reduction factors for the prestressed concrete and gusset region. The RP has carried out validation of this approach against the American code ACI349.1R-07 approach that is most closely aligned with ACI359. This is discussed further in Sections 4.6.4 and 4.6.6 below. While I consider that details of this validation work are lacking (see Section 4.4.1 above), I am content with the RP's use of ETC-C.

- The codes and standards used for the design of the internal containment liner (and its components) are outlined in Section 4 of Ref. 82. The design of the liner is in accordance with ACI359-17, but this does not include the concrete design requirements for the anchors, or the requirements for the brackets, including the polar crane brackets. The concrete design checks are undertaken in accordance with ACI349-13 and the brackets are designed in accordance with AISC N690-18. I am content that these are compatible with ACI359-17 and represent RGP.

In summary, for the purpose of GDA, I consider that the codes and standards the RP has applied for the internal containment design represent RGP, satisfy SAP ECS.3, and contribute towards meeting the intent of SAP ECE.2.

356. With respect to material specification, I note the following:

- The RP has specified C40/50 concrete grade for the raft under the BRX and C50/60 for the walls and dome of the internal containment, including the gusset region. I consider this appropriate and typical for post-tensioned construction in the UK.
- For the prestressing tendons, the RP is proposing 55C15 standard post-tensioned tendons in accordance with draft European standard prEN 10138. Despite this standard being in draft status, this standard is a widely used code. Notably, this standard is referenced in industry guidance such as the Concrete Society Technical Report 72 'Durable Post-tensioned concrete structures'. Therefore, I am content that this represents RGP. I note that the design and specification of the grout is considered a site-specific matter by the RP and I consider this appropriate, see also paragraphs 498.
- For the liner and anchor plates, the RP is proposing grade P265GH carbon steel. I note that the use of European material steel grades with US design codes (ACI359) for the liner has not been justified by the RP in Ref. 22. This will need to be reviewed at the site-specific phase.
- For the liner anchorage (studs and stiffeners) and brackets (polar crane brackets), S235JR grade steel is proposed. I do not expect there to be a concern with respect to the difference in steel grade between the liner and these components, but this will require further consideration within the weld specification during the site-specific phase.

In summary, aside from the two minor shortfalls noted above, I am satisfied that the material specification for the internal containment is in accordance with RGP and meets the intent of SAP ECE.16.

4.6.2 Analysis Models

357. The models used for the analysis and design substantiation of the internal containment are outlined in Ref. 23 and illustrated in Figure 7 of Annex 6 of this report. This use of models has been subject to expert review that is recorded in Section 6 of Ref. 28. From my assessment, I consider that the analysis models used by the RP are appropriate and allow conservatism to be demonstrated in a transparent manner consistent with SAPs ECE.12 and ECE.13.

358. During GDA Step 4, the RP developed the M1_Gusset model for design of the gusset area. As this model was developed, it became apparent that the M1_gusset model could ultimately replace the M1_model for design of the internal containment, at the site-specific phase. The RP confirmed this intention. I consider this is a positive commitment that can be assessed further, as a matter of normal business in the site-specific phase.

4.6.3 Loads

359. The load cases used for the design of the internal containment are outlined in Section 7 of the BoD (Ref. 50). Additional detail is provided in Section 7 of the SADR (Ref. 64). The key points from my assessment are as follows:

360. The self-weight of the internal containment is derived from the elements within the analysis model. The concrete density value includes a standard allowance for reinforcement (in accordance with EN1991-1-1) as well as the 6mm thickness of steel liner. The mechanical and electrical equipment attached to the internal containment is not modelled explicitly by the RP, rather, the self-weight of this equipment is accounted for with applied loads at the relevant nodes. This includes the hatches, brackets and polar crane. The load for the polar crane included in the M1 model assumes the crane is not carrying a load during operation. I am content with this for the global analysis of the internal containment, which is governed by accidental loading. The loads during testing and lifting are considered in the local design of the polar crane brackets, as discussed in Section 4.6.9. I note that the BRX internal structures are not included in the M1 model. Instead, the dead load is accounted for by applying a uniform pressure, as calculated from ANSYS Model 2 (where the internal structures are modelled). I consider this reasonable for GDA, as the raft under the BRX is designed using the results from ANSYS Model 2, where this aspect of the loading is more accurately represented. Creep and shrinkage actions are not modelled but are accounted for in the calculation of the prestressing loads, as well as within the post-processing of the design results.

361. There is no separate load case modelled for live loads. The RP acknowledges that construction loads should be considered but has declared this outside of the scope at GDA. I am content with this simplification.

362. The calculation of the prestressing loads, and how this relates to the modelling, is discussed in Section 4.6.5 below. I note that the maximum prestressing loads after post-tensioning are included in the construction and test load combinations. Furthermore, the minimum prestressing loads at the end of the service life are included in all other load combinations. I consider this reasonable for GDA, as prestressing is beneficial for the reinforcement demands which govern the design of the internal containment.

363. The pressure loads acting on the internal containment during normal operation are outlined in PCSR Chapter 10 (Ref. 132). This includes negative pressure in the annulus and pressure within the BRX ranging from 6kPa to -4kPa. In general, the operational pressure is much lower than that under accidental conditions: however, I note that the RP has considered the negative internal pressure for the design of the liner.

364. The normal operating thermal loads are defined in Section 7.5 of Ref. 50. Considering the further information obtained via RQ-UKHPR1000-0890, RQ-UKHPR1000-1082 and RQ-UKHPR1000-1162 (Ref. 6), I am content that these load cases are not expected to govern the design of the internal containment.

365. The test loads for the internal containment are defined in Section 7.7 and 7.8 of Ref. 50. I am satisfied these are consistent with the Construction and Testing Report (Ref. 75) and are in accordance with the requirements of ACI359-17.

366. The internal hazard loads considered for the internal containment are outlined in Section 7.1 of Ref. 33. Ref. 33 highlights that internal flooding can occur inside the BRX, and this is considered within the design of the internal containment as hydrostatic

pressure acting on the inner surface of the structure. High energy pipe failure results in large reactions at the penetration sleeves, which I discuss in more detail in Section 4.6.7. Internal fire can occur inside the BRX, and the RP has considered this within the concrete cover requirements, as opposed to applying a specific load in the analysis. See paragraph 354 above regarding the consideration of internal fire for the liner.

367. The only external hazard load considered for the internal containment structure is seismic loading, as outlined in Section 7.2 of Ref. 33. Seismic loading is considered in combination with the design basis accident (DBA) loading, and so it contributes to the governing design actions. The application of seismic loading to the internal containment analysis models is assessed in paragraphs 381 and 382 below.

368. The design basis accident loading governs the design of the internal containment. The pressure and thermal loads for the design basis accident were initially presented as stepped load-time functions. From this, four load cases were used in the analysis of the internal containment; based on 10 seconds, 2 hours, 10 hours and 100 hours after the accident. The RP used transient analysis to demonstrate that the maximum thermal moment at every position within the internal containment was found at the end of the analysis, i.e., in a single timestep (see Appendix F of Ref. 64). The internal temperature has reduced from a peak of 145°C to 95°C at this stage. Despite this reduction, the RP demonstrated that the steady state temperature profile through the thickness of the concrete is the most onerous case for the bending demand. The RP decided to combine this worst-case design basis accident thermal loading (steady state response at 95°C internal temperature) with the worst-case design basis accident pressure loading (0.42MPa relative pressure). I note that these conditions are unlikely to occur simultaneously and therefore consider this a conservative approach to envelope the internal containment demand in this way.

369. In addition to the thermal loads on the concrete, the concrete design must also consider the effects of the expansion of the liner. The temperature of the outer surface of the concrete will remain lower and will therefore act to restrain the expansion of the internal containment structure. The liner will be exposed to the high temperatures inside the BRX and will try to expand against the concrete, leading to compression in the liner and an equivalent pressure on the inside of the concrete face. The RP's methodology for calculating this load (outlined in Figure F-4.3-1 of Ref. 67) is as follows:

- The thermal load is applied to the liner model (with smeared thickness) with the nodes fully constrained and the reaction forces obtained.
- The reaction forces generated from the liner model are transferred to the M1 model. These reaction forces are combined with the design basis accident thermal, and pressure loads for use in design of the internal containment concrete. The nodal displacements of the liner nodes are then extracted from the M1 model.
- The nodal displacements from the M1 model are applied to the liner global model and the thermal loads reapplied to obtain liner stresses for use in design.

370. The RP describes this liner load on the concrete in Section 7.13 of Ref. 64. The magnitude of the load is verified using thin-walled pressure vessel theory, in Section 3.6 of Ref. 69. The RP confirmed that the equivalent pressure exerted on the concrete due to the liner expansion is equal to 24% of the design basis accident pressure load. I also noted that a liner yield strength of 320MPa was used in this analysis, and this is higher than the characteristic yield strength of the proposed liner steel grade P265GH. It remains unclear why this value has been used but I am content for the derivation of loads that this is conservative, even considering the effects of strain hardening of the liner.

371. For severe accident loads, the RP has included two scenarios in Section 7.15 of Ref. 50 that are considered within the ultimate capacity analysis, discussed in Section 4.6.12 below.

372. The load combinations for the design of the internal containment are outlined in Section 8 of Ref. 50, with further detail provided in Section 8.2 of Ref. 64. From my assessment, I am satisfied that these comply with the requirements of ACI359-17, noting that this includes the seismic loads combined with the design basis accident pressure and temperature loads.

373. In summary, I am satisfied that the loads defined and applied by the RP for the design of the internal containment satisfy ACI-359. I am satisfied that this meets the intent of SAP ECE.6 and ECE.13 and is adequate for GDA.

4.6.4 Analysis Methodology

Modelling

374. The overall analysis workflow for the design of the post-tensioned concrete is outlined in Figure F-4-1 of Ref. 23. With respect to modelling, the element types, meshing and boundary conditions have been subject to expert review recorded at Ref. 28. From my assessment, the key points are outlined below:

375. The M1 and M1_Gusset model both use 20 node solid elements in ANSYS to model the internal containment concrete. I am satisfied this is appropriate to capture the changes in geometry around the gusset, ring beam and hatches.

376. From my review of the mesh density and configuration, presented in Section 6.1.4 of Ref. 64, I note that the mesh of the internal containment wall is divided into five layers through the thickness, with a typical mesh size of 1m × 1m × 0.24m. I note that the dome is divided into four layers through the thickness, with a typical mesh size of 1m × 1m × 0.25m. The RP's validation of this mesh, presented in Section 3.4.2 of Ref. 69, compares the section forces used for design with a more refined mesh of 0.5m × 0.5m. This study (Ref. 69) indicated that large differences occur around the gusset and equipment hatch areas. The RP has therefore committed to use the M1_Gusset model and alternative design methods within the gusset region (discussed further in Section 4.6.6 below). The RP has also committed to use local models for the design of the hatches that I expect to have a further refined mesh. The differences found in other regions of the internal containment are less significant. For the purposes of GDA, I am content that the mesh density of the M1 model is reasonable in these regions.

377. With respect to boundary conditions, my assessment focused on the restraints on the underside of the raft, the cut section of the raft, and the cut section of the external containment. Regarding the fixed restraint to the underside of the raft in the M1-1 model (used for the analysis of the equivalent static seismic load cases), the RP investigated this to see if adopting very soft soil springs would affect the results. The results of this validation work are reported in Section 3.8 of Ref. 69, and indicate that for the base of the internal containment wall there was some sensitivity. In view of this, the RP committed to adopt the very soft (V_s of 150m/s) soil springs instead of the fixed restraint for the seismic load cases in both the M1-1 model and M1_Gusset-1 model. I am content with this approach for GDA.

378. For the cut sections of the raft, the validation studies are reported in Sections 3.4.3.3 and 3.4.3.4 of Ref. 69. These sections indicate that the peak stresses in the model are relatively insensitive to the extent of the raft modelled and that the free restraint was conservative for regions that were subject to the peak demands. I note that the peak stresses may not correlate to areas of maximum sensitivity and conclude that more extensive examination of the differences is needed. The study has now been

superseded by RP's development and use of the M1_Gusset model, which includes the whole of the common raft foundation, removing this issue. This change has been captured in the internal containment documentation.

379. The cut section of the external containment in the M1 model was fixed in the radial translational direction, idealising the restraint provided by the first suspended slab of the surrounding buildings. The RP's validation studies are presented in sections 3.4.3.1 and 3.4.3.2 of Ref. 69. These sections indicate that the fixed restraint was representative of the behaviour in the global ANSYS Model 2, and that this was conservative when compared to a free restraint. I am content with this approach to the modelling. The RP has committed to use the M1_Gusset model for the design of the internal containment at the site-specific stage. This will include the surrounding buildings, which I consider will more accurately represent the actual structural behaviour and constraints imposed by adjacent buildings.

380. For the M1_Gusset model, only the first two storeys of the surrounding buildings are modelled, with boundary conditions applied to the cut walls. The methodology for the boundary conditions is the same as other local models, with the displacements from the ANSYS Model 2 applied to the corresponding nodes in the M1_Gusset model. The validation work reported in Section 3.4.3.6 of Ref. 69 compares the cut section forces along the boundaries to validate the behaviour between the two models. The results indicate a 20% difference in bending moment under the design basis accidental thermal loading. Although I am content that this is not significant to the design of the gusset, this will require further consideration in future validation studies, see AF-UKHPR1000-0226 below.

Seismic Analysis

381. The seismic analysis approach for the internal containment is presented in Section 5.4.2.1 of Ref. 23, with further details provided in Section 7.10 of Ref. 64. The RP uses an equivalent static approach. This is based on the accelerations up the full height of the internal containment. The data is obtained from the ACS SASSI analysis of the buildings on the common raft foundation. I consider that equivalent static approach is acceptable if it can be demonstrated that contributions from higher order modes are insignificant for the local behaviour. The RP's validation study is presented in Section 3.5.2 of Ref. 69. This found that the stresses in the ring belt (at the base of the dome) and in the dome region were underestimated using the equivalent static approach. Therefore, the RP has committed to revise their design approach at the site-specific stage. I note that the RP has not quantified the impact of these stresses on the design. From Section 3.5.1 of Ref. 69, the global seismic demand on the internal containment is clearly conservative, with higher shear cut-section forces in the M1 model, compared to the ACS SASSI analysis. I judge that RP's revised approach should therefore only have a local impact on the design of the internal containment. This future improvement is captured in assessment finding AF-UKHPR1000-0224.

AF-UKHPR1000-0224 – The licensee shall, as part of the site-specific design, demonstrate that the seismic analysis approach for the internal containment captures the local response in a conservative manner. This should include, but not be limited to, the ring belt and dome region of the internal containment.

382. With respect to damping values for the internal containment, the RP has used 'Response Level 2' damping values, in accordance with Section 3.2.2 of ASCE4-16 for all structures. These have been applied as material damping in the ACS SASSI model. This equates to 5% damping for the internal containment as a prestressed concrete structure. I consider that the internal containment will remain essentially elastic under design basis conditions. This would indicate that 'Response Level 1' damping values

would be appropriate for seismic loads, in the absence of accident loads. I judge that higher levels of damping may be appropriate under the governing load combination of accidental plus seismic loading. The RP has committed to use the iterative approach at the site-specific stage. From my assessment, I consider that this may reduce damping and thus increase the seismic demands. This further work is captured in assessment finding AF-UKHPR1000-0217 above.

Thermal Analysis

383. The thermal analysis of the internal containment was carried out with a thermal version of the M1 model, using a different element formulation, see Ref. 23. The boundary temperatures for each thermal load case were confirmed and are reported in Table T-F-5 of Ref. 64. From my assessment, I am satisfied that this approach for deriving the thermal load case is adequate for GDA.

384. For the analysis of the structural response to thermal loads, the RP's approach for the internal containment is similar to the general approach described in Section 4.4.6 above. Stiffness and load factors are applied under operational and design basis accidental conditions. The RP has confirmed in Table T-F-3 of Ref. 64 that different stiffness and load factors are applied for the internal containment and gusset above the common raft foundation. The analysis and design of the raft under the BRX follows the same methodology as the other reinforced concrete structures. From my assessment, it remains unclear whether this approach is appropriate for the stress state of the common raft foundation under thermal loading. It is uncertain whether the central portion of the raft is put into compression as it expands against the surrounding sections of the common raft foundation. The RP has committed to validating this cracking assumption at the site-specific stage. This requirement for further work is captured in assessment finding AF-UKHPR1000-0225.

385. The load reduction factor of 0.5 that is applied to the internal containment and gusset design is based on Clause 1.4.4.1 of ETC-C for the prestressing wall of the containment. To demonstrate that this load factor was appropriate for the level of prestressing in the proposed design (that is expected to remain essentially elastic under design basis conditions), the RP undertook further validation (Appendix G of Ref. 64). The results of this validation indicate that a factor of 0.5 on the design basis accident thermal load case is conservative. However, I note that the validation was undertaken at locations close to non-standard zones, and so does not represent the typical situation in the standard zone. Nevertheless, the application of the 0.5 factor in the standard zone follows ETC-C and I am satisfied that this is appropriate for the purposes of GDA and that further validation for the standard zone can be presented in the site-specific phase. The requirement for further validation work is captured in assessment finding AF-UKHPR1000-0225.

386. The above points relating to the thermal analysis of the internal containment are consolidated in assessment finding AF-UKHPR1000-0225 below.

AF-UKHPR1000-0225 – The licensee shall, as part of the site-specific design of the internal containment, validate the application of the thermal reduction factors considering the structures stress state. This should include but not be limited to the common raft foundation under the reactor building and the internal containment standard zone.

387. From my review of the methodology presented in Appendix G of Ref. 64, I note that the methodology does not appear to follow an iterative approach. An iterative approach is required to capture stresses as a result of a restraint which may occur at the junction with the gusset. Therefore, I was not satisfied that the ETC-C factor could be applied to

the gusset without further validation, as this case was expected to be governed by shear, as opposed to flexure. This is discussed in Section 4.6.6 below.

Validation

388. The validation process for the internal containment analysis is presented in Ref. 69, with an overview of the process illustrated by Figure F-2.2-1 of that report. From my assessment, I note that the studies undertaken focus on the inputs and outputs to the M1 model, with little validation presented for the other internal containment models. Although the RP has sufficiently met the intent of SAP ECE.15 with respect to the M1 model, my expectation is that an equivalent level of validation is undertaken for the other models, including the M1_Gusset model, at the site-specific stage. Furthermore, the RP has not used the non-linear ABAQUS model to validate the results of the M1 model. Although I do not consider this to be an essential for GDA, as per the intent of SAP AV.2 it could strengthen the RP's assumptions relating to non-linear behaviour under thermal loads and within the gusset.

389. This requirement for further validation is captured within the overarching assessment finding AF-UKHPR1000-0226 that is introduced in paragraph 407 below.

Sensitivity Studies

390. The sensitivity studies that have been performed for the internal containment include the structural mesh sensitivity studies; see paragraph 376 above. These sit alongside the seven sensitivity studies supporting the ACS SASSI seismic analysis, discussed in Section 4.4.5 above. Some of these sensitivity studies have resulted in commitments to revise the seismic analysis methodology at the site-specific stage; see assessment finding AF-UKHPR1000-0217. The commitments relevant to the internal containment are discussed below:

- Time Histories: The RP has committed using a set of 5 time histories at the site-specific stage, which may affect the seismic demand on the internal containment.
- Damping: The RP has committed to an iterative approach to damping at the site-specific stage which may result in increased seismic demand on the internal containment.
- Concrete Stiffness: The effect of cracking on the member demands of the internal containment was not presented, but it is expected that the uncracked condition is representative and conservative for the design of the internal containment. The effect of cracking is more significant for displacements and floor response spectra, so the commitment to revisit the cracking assumptions at the site-specific stage is expected to be more significant for the supporting SSCs within the internal containment.
- Mesh Size: This is not expected to be significant for the internal containment design with the quasi-static seismic loading but may need to be considered when the seismic analysis approach is updated; see assessment finding AF-UKHPR1000-0224 above.

391. In summary, for the purpose of GDA, I am content that the use of sensitivity analysis for the internal containment accords with SAP ECE.14.

4.6.5 Post-Tensioned Concrete Analysis and Design

392. The analysis and design of the post-tensioned concrete in the internal containment is in accordance with ACI359-17. The RP's acceptance criteria are allowable stresses for the concrete, reinforcing steel and prestressing tendons, under both service and factored load combinations, as outlined in Section 9 of Ref. 50. These are post-

processed in a similar way to the reinforced concrete buildings, with additional checks undertaken for the non-standard zones. I am content with this approach.

Prestressing Actions

393. The RP's calculation of the prestressing action within the analysis and design workflow is outlined in Ref. 30 and illustrated by Figure F-8.4-3 of that report. This includes the calculation of the prestressing actions, inputs and transfer to the analysis models, and the outputs to the design software.

394. The RP's calculation of the prestressing losses at the end of the design service life is reported in Ref. 81. This includes calculation of friction losses, anchorage slip, elastic shortening, shrinkage of concrete, creep of concrete and stress relaxation of tendons. The calculations for friction losses and elastic shortening are undertaken in accordance with ACI423.10R-16. The RP's calculations for time dependant losses associated with shrinkage, creep and relaxation are undertaken in accordance with EN1992-1-1 and EN1992-2. The RP confirmed that these calculations are undertaken for every tendon, based on its individual geometry. The RP provided an example in Appendix D of Ref. 64. From my assessment, whilst I consider that the methodology for calculating losses is adequate for the purpose of GDA, I highlight that the losses will be dependent on the prestressing sequence, which is outside of the scope of GDA.

395. To account for the effects of concrete creep, the RP has increased the prestressing load applied to the analysis model to account for the additional compressive stress in the reinforcement resulting from concrete creep. I note that there is ambiguity and a possible contradiction with Section 8.4.2 of Ref. 30, which states that the additional compressive stress in the reinforcement due to concrete creep is added manually, as opposed to being an output from the M1 model. This alternative method allows the additional stress to be conservatively ignored if the reinforcement is in tension and represents the situation at the beginning of the design life. I consider this to be a more appropriate method that avoids overestimating the amount of prestress in the design. However, it is unclear why the equivalent temperature drop (shown in Figure F-8.4-3 of Ref. 30) includes the additional stress, as this appears to overestimate the amount of prestress in the design. I consider that this apparent contradiction in methodology is a minor shortfall that can be resolved as normal business in the detailed site-specific design.

396. Furthermore, I note that the maximum prestressing actions (occurring immediately after construction, i.e., prior to creep) are considered within the construction load case, whilst the reduced prestressing actions including creep losses are considered in all other combinations. I judge this to be appropriate for the purpose of GDA

Modelling of Prestressing

397. The prestressing actions on the concrete are calculated using a version of the M1 model which includes the tendons modelled as one dimensional (1-D) elements (M1+tendon model), see Ref. 28 for further detail. Within this model, the tendons are constrained to the nodes of the solid concrete elements and a thermal strain is applied equivalent to the prestress with losses, as discussed above. The resulting prestressing actions that act on the concrete are extracted from the M1+Tendon model as nodal displacements. These displacements are applied to the M1 model for combination with the other actions. This process is described in Section 7.4 of Ref. 64. From my review, I am satisfied with the accuracy of the tendon geometry in the M1+tendon model compared with the drawings, including the deviations of tendons around penetrations. Furthermore, I particularly commend the RP on their thorough validation of the significance of frictional resistance between the tendons and ducts, and the consideration of how this is affected by the curvature of the wall and tendon deviation. However, this work has not been reported and should be incorporated to record the

validation work. I consider this a minor shortfall that can be resolved as normal business in the detailed site-specific design. From my assessment, two points are worth noting and are highlighted in the subsequent paragraphs.

398. As the tendon elements are constrained to the nodes of the concrete solid elements, I note that the local strains in the 1-D elements representing the tendons may vary along the length of each tendon. This is a function of the relative stiffnesses of the concrete and tendons, which is affected by openings, thickenings and tendon spacing. The RP investigated the significance of this by analysing a straight section with varying concrete thickness; see Section 3.7.3 of Ref. 69. I note that the results do not show a variation in prestressing force, which would be expected if the tendon elements were constrained to the concrete. I consider that this requires further validation in the site-specific phase in accordance with SAP ECE.15. This is recorded by assessment finding AF-UKHPR1000-0227 below.

AF-UKHPR1000-0227 – The licensee shall, as part of the detailed design, justify the modelling approach for the internal containment post-tensioned tendons. This should include, but not be limited to, the tendon element constraint.

399. Furthermore, I note that as the applied temperature loads have not accounted for the stiffness of the tendons, the prestressing approach underestimates the stress in the tendons. The RP's validation of this in Section 3.7.2 of Ref. 69 shows that the peak stress in the tendons is underestimated by 6%. The RP has committed to incorporate the stiffness scaling into the analysis methodology. The documentation associated with this has yet to be updated, namely Ref. 23 and Appendix B of Ref. 64. I consider this a minor shortfall that can be resolved as normal business in the detailed site-specific design.

Post-Processing

400. The post-processing of the internal containment results follows a similar process to the other reinforced concrete buildings, described in Section 4.4.8. Since the internal containment M1 and M1_Gusset models have solid elements, the internal forces are calculated by integrating the results through the thickness of the wall. This is reported in Section 8.1 of Ref. 64. The resulting section forces and moments are input to the design software to calculate the stresses in the concrete, reinforcement and prestressing tendons. This process assumes a linear stress distribution through the thickness of the wall i.e., plane sections remain plane. The validation of this assumption is reported in Section 3.10 of Ref. 69. This confirms that the assumption is valid within the standard zone, away from geometrical discontinuities. The strength design of the internal containment uses the REINCAL design software discussed in Section 4.4.8. The software assesses the results against the allowable stresses based on the equations in ACI359-17 and is reported in Section 8 of Ref. 30. I am content with this approach.

401. From my assessment, my main query related to how the prestressing actions were considered within the design checks to calculate the stresses in the concrete, reinforcement and tendons. The RP confirmed that an average stress is considered for each type of tendon and that REINCAL does not consider the actual stress in the tendon at the location of the check. I consider this to be conservative towards the middle of the tendon where friction losses are above average, but unconservative at the ends of the tendons. The RP proposed to adopt lower utilisation limits for the prestressed tendons in some regions, as shown for the horizontal tendons in Figure F-8.4-2 in Ref. 30. Whilst I am satisfied that this should be conservative for the purpose of GDA, I do not consider this good practice for future design phases, as the actual

utilisations along the tendons cannot be assessed. This improvement required is captured in assessment finding AF-UKHPR1000-0228.

402. With respect to the additional compressive stress in the reinforcement due to concrete creep (discussed in paragraph 395 above), the RP confirmed in response to RQ-UKHPR1000-1526 (Ref. 6) that the additional stress is only added when the reinforcement is already in compression, ensuring it is only considered when conservative to do so. I am content with this approach.

403. The tangential shear checks implemented were modified by the RP to be in accordance with ACI359-17. I note that this reduced the maximum permissible shear force by 40%, although the RP claimed that this did not impact the design. Separately, I note that the calculated tangential shear reinforcement in the standard zone has reduced significantly during Step 4. The RP stated this was due to the revised approach for the design basis accident thermal loading, discussed in paragraph 368 above. I remain unclear why this would be the case if the revised approach is conservative, as it is claimed to be in Appendix F of Ref. 64. Further explanation for this substantive reduction is required as noted in assessment finding AF-UKHPR1000-0228.

404. The above points are consolidated in assessment finding AF-UKHPR1000-0228 below.

AF-UKHPR1000-0228 – The licensee shall, as part of the detailed design, resolve the following regarding the post-processing of the internal containment analysis results: • Present the actual utilisations for the post-tensioned tendons that explicitly considers the losses along the length of the tendon rather than using an average prestress. • Justify the reduction in tangential shear reinforcement in the standard zone.

Non-Standard Zones

405. The RP identifies in Figure F-5.1-1 of Ref. 64 a number of non-standard zones within the internal containment including the ring belt, ribs, hatches and gusset. These non-standard zones are understood to represent the discontinuous regions (D-regions), as defined in ACI318 and discussed in paragraph 236 above. I anticipated strut and tie methods to be used for the design, in accordance with Ref. 25. However, the strength design for both the equipment hatch and gusset (presented in Sections 9.3 and 9.4.6 of Ref. 64) uses the same design process as the standard zone, which assumes a linear stress-strain relationship. The RP has carried out further validation of this assumption, reported in Section 3.13 of Ref. 69. From my review, I am satisfied that this demonstrates that the linear stress-strain assumption is reasonable for all regions apart from the gusset; see Section 4.6.6 below.

406. The design of the region around the equipment hatch is reported in Section 9.3 of Ref. 64. From my assessment, I consider that additional checks will be required for the transfer of loads between the sleeve and the concrete, noting that the RP has only presented equivalent checks for the high energy pipe penetration at GDA. Furthermore, the RP confirmed that a local model of the equipment hatch would be used for the design basis conditions at the site-specific stage, and that similar local models would be used for the emergency hatch and personnel hatch. No further information on the extent of these local models has been provided during GDA. The use and integration of local models into the overall modelling approach is an area that will require detailed consideration during the site-specific design. This and the need for comprehensive validation is highlighted in assessment finding AF-UKHPR1000-0226.

407. I note from Section 9.3.3 and Figure F-9.3-6 of Ref. 64 that the reinforcement and concrete stress ratios (utilisations) around the equipment hatch are reported as exceeding 1.0. I expect that this is related to lower prestressing where the tendons deviate around the opening. Furthermore, I note in the M1 model that the mesh is relatively coarse and not well defined around the equipment hatch, which may affect the results. For the site-specific phase, I consider that the licensee will need to carry out more detailed modelling and validation to justify the behaviour of this area. This further work is captured in assessment finding AF-UKHPR1000-0226 below.

AF-UKHPR1000-0226 – The licensee shall, as part of the detailed design, justify the overall finite element modelling approach for the internal containment. This should include, but not be limited to, the use of both global and local models, the extent of these models and how they interface and interact, and the validation applied.

Anchorage Zones

408. The detailed design of the anchorage zones for the prestressing tendons is presented in Section 10.2 of Ref. 64. The design of these regions is not covered in ACI359-17, and so the RP has chosen to follow the requirements of EN1992-1-1. I consider this to be appropriate RGP consistent with the post-tensioning systems and reinforcing detailing used in the UK. From a sample assessment, I have gained sufficient confidence that the RP has specified and applied the formulae appropriately.

Tendon Failure

409. The effect of tendon failure has been assessed by the RP and is presented in Ref. 133. From my assessment, the RP's methodology has satisfactorily addressed the basis of re-anchorage length, the modelling of re-anchorage and the re-anchorage demand on perpendicular reinforcement. However, the RP has provided limited evidence to demonstrate that the most critical locations and load combinations have been considered in the tendon failure analysis of the internal containment. I consider this a minor shortfall that can be resolved as normal business in the detailed design.

410. The RP has decided to set the maximum number of concurrent tendon failures to three. The RP demonstrates in Ref. 133 that both the strength capacity and monitoring system are sufficient under this scenario to provide the margin needed before failure is detected. I am content that this approach is in line with the guidelines in ONR TAG 20 (Ref. 4). However, the arrangement of the monitoring devices, which will define its resolution and will be determined from the tendon failure study, is outside of the scope of GDA. From my review of the methodology, I note that no discussion has been provided by the RP on the expected change in strain of the internal containment due to creep and shrinkage, as opposed to tendon failure. This may not be uniform across the internal containment. This may create noise relative to a theoretical time-dependent strain state which could impact the effective strain gauge sensitivity. The design and operation of the monitoring system in the site-specific phase will need to allow for changes in strain (due to concrete creep and shrinkage) to correctly identify tendon failure. I consider that this can be resolved as normal business in the detailed site-specific design.

Design Margin

411. The utilisations for the concrete, reinforcement and tendons within the standard zone and the governing load cases are presented in Ref. 64. The information presented is clear and demonstrates the typical utilisations, allowing the reader to interrogate the actions causing high utilisations in sample areas. However, I note that the results and narrative presented in Section 10.2 of the DSR Ref. 86 provides little assurance that sufficient design margin is provided at GDA. For example, the reinforcement in the

standard zone is reported to have a typical utilisation of 0.8, with a peak utilisation of 0.99. My assessment of the results presented in Ref. 64 indicates utilisations up to 0.7 within the standard zone, with non-conforming peaks only locally around the openings. I expect these areas to be analysed in more detail using local models.

412. Furthermore, the RP has provided limited narrative within the documentation to explain the behaviour and governing actions on the internal containment. From an expert review (Ref. 28) of the results that the RP presents in Ref. 64, I am content that an adequate design margin exists within the standard zone of the internal containment. I note that utilisations exceeding the acceptance limit of 1.0 are reported for the equipment hatch, gusset, and high energy pipe penetration. However, these areas require more detailed consideration, as noted in paragraphs 405-407 above.

413. In summary, my assessment has highlighted a number of matters: however, I am content that the RP has provided a sufficient demonstration of the internal containment post-tensioned concrete analysis and design approach that meets the intent of GDA.

4.6.6 Gusset Analysis and Design

414. The design of the internal containment gusset region was an area of focus for my assessment, as it is an example of a non-standard zone which requires the application of alternative design methods. The gusset is intrinsic to multiple load paths. In addition to actions from the internal and external containment, actions are exerted on the gusset from the internal BRX structures arising from seismic and thermal effects. The modelling of the M1_Gusset model is reported in Section 11 of Ref. 63. The design of the gusset is reported in Section 10.5 of Ref. 64 and substantiated in the internal containment DSR (Ref. 86). I consider that this separation of the reporting reduces the clarity of the safety case for the gusset (see SAP SC.4), which is a key element within the civil engineering design. I am content this is a minor shortfall that can be improved upon in future stages as normal business.

Modelling

415. There is extensive overlap of the areas modelled using ANSYS Models 1 and 2, the M1 model and the M1_Gusset model. The RP's modelling strategy with regards to the interfaces between the internal containment, gusset and common raft foundation has evolved during the course of GDA. It remains the case that no areas are being designed using the envelope of multiple models. The extents for which each model is being used in the design is clarified in Figure F-5.1-2 of Ref. 64. This indicates that the M1_Gusset model is used to design the portion that would be considered as a discontinuity (in accordance with ACI349) between the internal containment, external containment and the common raft foundation. However, design checks using the M1_Gusset model have only been presented for the region of the gusset above the common raft foundation. I judge that this discrepancy is not significant as long as standard detailing rules are followed to demonstrate that the shear capacity is extended into the internal and external containment and common raft foundation. I am content that this minor shortfall can be resolved as normal business in the site-specific phase.

Seismic Analysis

416. With respect to the seismic analysis, the RP confirmed that the equivalent static approach is used in the M1_Gusset model, just as it is used in the M1 model. The maximum accelerations for each building on the common raft foundation are extracted from the seismic analysis (within ANSYS Model 1) and applied as an equivalent static load to the ANSYS Model 2. The nodal displacements are then applied to the M1_Gusset model. I am satisfied that this approach is conservative for the purpose of

GDA in line with SAP ECE.13. I expect that the approach will be modified at the site-specific stage as the M1_Gusset model is developed further to replace the M1 model.

417. The seismic load path for the internal structures includes resistance provided by base friction, bearing resistance against the upstand key in the centre of the BRX and bearing resistance against the gusset. For GDA, the RP has ignored the sliding resistance from friction and the upstand key and designed the gusset for the full seismic force from the internal structures. This is reported in Section 10.5.5 of Ref. 64. The force is then applied as an equivalent static pressure to one side of the gusset. I am content this simplification is conservative for the gusset and is in accordance with SAP ECE.13.

Thermal Analysis

418. For the thermal analysis, thermal reduction factors are used for the gusset region. The RP presented the validation of these factors on 11th of December 2020 and 8th January 2021 (see Ref. 10). This demonstrated that the thermal gradient through the gusset was primarily resisted by the outer hoop reinforcement, as opposed to the vertical reinforcement. The results from a non-linear calculation method demonstrated that the load factor of 0.5 was conservative to calculate the stress in the outer hoop reinforcement. This validation has not been reported in the RP's submissions, and the details of the non-linear calculation method are unclear. I understand the method uses the software 'Grather_tt' which has been developed by the RP's TSC and was initially described as a pseudo non-linear methodology in accordance with RCC-CW DCONC 4222. However, from my review I note that the approach did not appear to be fully consistent with this code. As a result, the RP clarified in response to RQ-UKHPR1000-0890 (Ref. 6) that the approach draws from RCC-CW, ACI349.1R-07 and fib 2010, bulletin No. 46(Annex 2). The RP confirmed that the thermal reduction factor is only applied to the design basis accident thermal load case 'TL', and only reduces the bending moment. The axial and shear forces are not affected by this aspect and, from my assessment, I am content with this claim.

419. With respect to the internal structures, the thermal analysis model includes these structures to calculate the pressure on the gusset resulting from their expansion. The base of these internal structures is assumed to be constructed against the gusset, so a fixed constraint is used in the analysis. The RP ignores the base friction in the thermal analysis, and the gusset is designed for the full reaction from the internal structures. Overall, I consider the RP's approach for considering the thermal expansion of the internal structures in the design of the gusset to be conservative, in line with SAP ECE.13.

420. In summary, I am satisfied that the thermal analysis of the gusset region is in accordance with RGP. However, I note that the validation of the thermal load reduction factor has not been adequately reported during GDA. I consider this a minor shortfall that can be resolved as normal business in the detailed site-specific design.

Load Paths

421. The analysis results from the M1_Gusset model are presented in Section 10.5.7 of Ref. 64. These results demonstrate the load path that is resisting the design basis accidental thermal and pressure loads. Although the narrative is lacking in Ref. 64, it is clear that the design basis accident pressure load is less than the prestress on the internal containment, so the net effect remains an inward shear on the gusset. This is carried through the gusset and is resisted by the common raft foundation under the BRX. This behaviour is consistent with the stress distribution through the top of the gusset, provided in response to RQ-UKHPR1000-1430 (Ref. 6) which shows high bending stresses in the external containment. I expect the licensee to provide more

comprehensive narrative to explain this behaviour in the site-specific phase, as part of normal business.

422. The dominant load paths resisting the thermal expansion of the internal containment are less clearly articulated in the RP's submissions. The RP concludes that this load is mainly resisted by the surrounding buildings, although there also appears to be a contribution to the resistance from the common raft foundation and hoop action within the gusset. This lack of clarity relates to the justification of the thermal reduction factor for the gusset region. Nonetheless, the RP has confirmed that the internal containment actions are included in the ANSYS Model 2. The RP has also confirmed that any load path through the surrounding buildings would be captured within the design of the connecting elements, to avoid losing these forces in the interface between models. The licensee should explicate this in more detail in the site-specific phase; I consider this normal business.

423. From my assessment, I also note that the shear load path through the gusset is sensitive to the underlying soil stiffness. This indicates that the construction staging of the internal containment, external containment and surrounding buildings could affect the load path on the gusset. The RP has committed to consider this at the site-specific stage and I am content this is normal business.

424. Overall, for the purpose of GDA, I am content with the RP's examination of the load paths. However, at the site-specific phase I expect a more thorough evaluation of these aspects mentioned above to be presented, as per the intent of SAP ECE.12. I am content that this represents normal business.

Strength Design

425. The RP's strength design approach involves splitting the gusset into sub-sections, where each sub-section has an approximately linear stress distribution that can be checked with REINCAL, see Figures F-10.5-21 and F-10.5-41 of Ref. 64. The methodology for checking the reinforcement demand within each sub-section is presented in Sections 10.5.9 to 10.5.13 of Ref. 64. The axial and bending demand on the vertical reinforcement is considered separately to the additional demand, due to radial shear. From my assessment, I note that these do not appear to be combined correctly. For example, the stress ratio in the vertical reinforcement due to radial shear is reported as 1.09 for Section E-F which is combined with membrane plus bending stress ratio of 0.03, shown in Table T-10.5-23 of Ref. 64. However, the reinforcement stress ratio due to membrane plus bending for layer 9 is reported as 1.9 elsewhere, as shown in Table T-10.5-25 of Ref. 64. These checks are understood to be on the same reinforcement layer, shown in Figure F-10.5-42 of Ref. 64. The checks indicate the demand is almost three times the capacity of the reinforcement provided. The RP has stated that the reinforcement in layer 9 could be doubled from 2D40 @ 200 to 4D40 @ 200, but this would still not meet the combined demand without averaging. With this solution being proposed, I consider that the strength design method is overly conservative and is leading the RP to a solution that has potential constructability implications. Underpinning my conclusion, I note that separate validation of the demand on the vertical reinforcement was presented by the RP's TSC at the RQ technical progress meeting #04 (See Table 2 above and Ref. 10). This presentation indicated that the current 2D40 vertical reinforcement is sufficient: however, the RP has not included this check or articulated this within the GDA submissions. This is recorded by assessment finding AF-UKHPR1000-0229.

426. Design checks on the concrete are presented in Table T10.5-21 of Ref. 64. I note that the concrete stress results show high utilisations when combined with membrane plus bending stresses. I note that the RP's methodology is also not consistent with strut and tie models, since the full width of the sub-section is considered, without accounting for the finite size of the compression struts. Since high concrete stress ratios are reported,

it is important that the RP improves this methodology for future design development. However, from my assessment I note that the global shear demand on the gusset indicates that the concrete should not be highly stressed. Furthermore, I note that the results presented by the RP do not align with the information presented by the RP's TSC during Step 4 that indicate more realistic concrete stresses. The RP has not included these calculations or explained this detail within the GDA submissions. This is recorded by assessment finding AF-UKHPR1000-0229.

427. In summary, I am content with the sub-section methodology reported for the gusset. However, the narrow sub-sections generate high strut and tie demands and a revised approach may lead to improved results. The results reported in Ref. 64 show very high utilisations, with no clear commitment for how these will be resolved at the site-specific stage. Furthermore, the RP has presented work carried out by their TSC that does not appear to validate the results and this work has not been reported in the GDA submissions. Further work is required in the site-specific phase to resolve these aspects as captured by assessment finding AF-UKHPR1000-0229.

AF-UKHPR1000-0229 – The licensee shall, as part of the detailed design, refine and validate the strength design methodology for the internal containment gusset to demonstrate that:

- the methodology for combining demands on the vertical reinforcement from shear with the axial and bending demands is adequate; and
- the methodology for checking concrete stresses within the gusset sub-sections is in accordance with relevant good practice for strut and tie models.

428. Overall, I am content with the RP's progress with the design of the gusset during GDA. The RP has gone to a level of detail beyond that presented in previous GDA's which is to be commended. This work has enabled my assessment to highlight areas that require further consideration and improvement that should de-risk future design development. Therefore, I am content that the RP has fulfilled the purpose of GDA with respect to the internal containment gusset.

4.6.7 Penetration Analysis and Design

429. The internal containment design includes several penetrations to accommodate hatches and pipe sleeves. It is expected that local models will be used to assess the local effect of specific load cases and to undertake detailed design checks. To demonstrate this for GDA, the RP chose to focus their reporting on the area surrounding the high energy pipe penetration. This is reported in Section 10.1 of Ref. 64. The penetration for the high energy pipe is included in the M1 model but the application of accidental loading is simplified. I note the RP has used a local model to simulate the loading and structural form more accurately.

430. From my assessment, I note that the transfer of data between the local penetration model and the global M1 model is the same as the general process shown in Ref. 25. The application for the high energy pipe penetration is illustrated in Figure F-10.1-1 of Ref. 64. The Figure shows that the mesh of the local model is much more refined than the M1 model, and the annotations indicate how the boundary conditions are automatically defined, based on the nodal displacements in the M1 model. The load application is described in Figure F-10.1-8 of Ref. 64. From my assessment, for GDA I am content with the modelling approach adopted.

431. For the strength design, I note that the bending and shear force on the pipe sleeve is predominantly resisted by bearing on the concrete. Using the results from the local model, the RP found that the concrete compressive stresses were 10% higher than the allowable stress. I note that this result is based on a conservative envelope of all the

accidental pipe reactions that the RP has assumed within the local models. The RP has committed to refine this model analysis and consider the individual load cases at the site-specific stage. I expect this will reduce the demand; however, I am uncertain whether this will counter the full 10% overstress. The axial force on the sleeve is resisted by flanges welded to the outside of the sleeve, shown in Figure F-10.1-2 of Ref. 64. Regarding transverse shear in the concrete, the RP demonstrates that the axial force was justified, using the approach for concrete breakout strength of anchors in tension from Clause D.5.2 of ACI318. Additional transverse reinforcement is provided within the concrete failure prism to meet this demand. The torsional force on the sleeve is resisted by ribs welded to the outside of the sleeve, also shown in Figure F-10.1-2 of Ref. 64. The RP has demonstrated that no reinforcement is required to resist the torsion outside of the ribs, in accordance with the relevant ACI359-17 calculation. In summary, I am content with the strength design methodology applied to this local model but note that further work is required to justify the concrete compressive stresses. I record this in assessment finding AF-UKHPR1000-0230 below.

AF-UKHPR1000-0230 – The licensee shall, as part of the detailed design, refine the analysis and design approach for penetrations to demonstrate the internal containment design is adequate under accidental loading arising from high energy pipe failure.

4.6.8 Liner Analysis and Design

432. The analysis and design of the internal containment liner and its components is reported separately to that for the post-tensioned concrete. I note that the structuring of the liner reports also differ. The basis of design and analysis methodology for the liner are presented in Ref. 82, while the design results are reported in Ref. 67. From my assessment, I note the RP's areas of scope excluded from GDA; these are as follows:

- The analysis and design of the baseplate of the internal containment liner. I am content that this is not a critical area for the design.
- The weld details of the internal containment liner. I expect these to be full strength welds in accordance with ACI359, so I am content this should not have a significant effect on the behaviour of the liner. I note that the welds in areas of stress concentrations and / or areas with plate yielding will require special attention at the site-specific stage. These welds may be in critical locations, such as at the joint between the baseplate and the gusset.
- The substantiation of the liner under construction loading. I am content that the loads appear unlikely to govern the design of the liner, based on evidence presented by the RP from FCG3; see Appendix G of Ref. 67.

433. The modelling approach for the liner is illustrated by Figure F-4.3-1 of Ref. 67. This includes global and local liner models with non-linear shell elements, each developed in ANSYS. The liner model and the process used to generate the load case for the post-tensioned concrete design is described and assessed in paragraph 369 and 370 above.

434. The version of the global liner model used for the design of the liner itself only models the 6mm thickness of the liner, in order to obtain peak stresses. The boundary constraints for this model come from the M1 model. The loading due to the liner expansion in the M1 model is based on fixed constraints, and the RP does not propose to iterate the displacements. I am satisfied this is conservative for the concrete, and I do not expect this consideration to be significant for the design of the liner.

435. From my assessment of the local liner model, I note that the RP has modelled the influence of the stiffeners with springs, whilst nodal restraints represent the influence of the studs. I note that nodal restraints are applied in the plane of the liner, as well as out of the plane, which would reduce the shear force in the springs representing the stiffeners. The RP investigated the significance of this by removing the nodal restraints and confirmed that the results indicated very little difference in the demands when the restraints are removed. The RP also claimed that limited demand on the stiffeners is expected, as the liner will carry arching forces due to the axisymmetric geometry. The RP concluded that their methodology is suitable. From my assessment, I accept that this may be the case in the standard zones, and I highlight that the shear demand of the stiffeners will be higher next to the hatches where the arching forces cannot develop. These non-standard regions are not considered by the RP during GDA. The RP will require a modified methodology using appropriate representation of the restraints to address this at the site-specific stage. I consider that this a minor shortfall that can be resolved as normal business in the detailed site-specific design.

436. The liner and its anchors are designed in accordance with ACI359. Allowable strains are the main acceptance criteria for the liner under service and factored load combinations, as reported in Section 12.1 of Ref. 82. The results reported in Section 5.8 of Ref. 67 show that the highest strains occur around the equipment hatch, under the combined design basis accident plus seismic load combination. I note that these results are less than half the allowable strain. I am content that the thickness of 6mm is driven by practical constraints during construction, which is consistent with experience from other similar UK projects. I note that as the liner is used as permanent formwork, it will attract compression as the concrete is prestressed during construction and will continue to attract compression as the concrete creeps and shrinks in a similar way to the reinforcement. The RP presents results reported in Section 5.8.3 of Ref. 67 that show the maximum compressive strain under operational conditions is only 0.62×10^{-3}. From this, it appears that the additional compressive strain due to concrete creep and shrinkage has not been included, which will increase the compressive strain results. Despite this omission, I am content that there is sufficient margin within the design to accommodate additional strains in the order of 1×10^{-3}. I consider that this is a minor shortfall that can be resolved as normal business in the detailed site-specific design.

437. The RP's approach for the design of the liner anchors follows the guidance in ACI359, reported in Section 12.2 of Ref. 82. I note that additional requirements for the liner anchors are outlined in Clause CC-3810 of ACI359. This clause states that stud anchors shall be designed to fail before tearing the liner. The RP highlighted in RQ-UKHPR1000-1274 (Ref. 6) that physical tests undertaken for FCG3 and additional studies have demonstrated that the stud anchors failed before the concrete, and so the performance of the liner is not compromised. The RP has committed to undertake additional tests on the shear and tensile failure of the stud anchors at the site-specific stage. I am content with this and consider this to be normal business.

438. The sleeves around penetrations are part of the civil engineering domain but have not been sampled or assessed in detail during Step 4. The RP will need to consider the local loading effects on these penetration sleeves, captured in local models. I am content that this is normal business.

439. In summary, for the internal containment liner, I am satisfied that the RP has applied RGP for GDA and subject to the points above being addressed, I am confident the liner design can be fully substantiated in the detailed site-specific design.

4.6.9 Polar Crane Support Structures

440. There are several anchor plates that are integral to the internal containment liner that are required to support SSCs. The biggest of these are the polar crane brackets which the RP included as part of the GDA scope. These are illustrated in Figure F-3.6-1 of

Ref. 82, the requirements and design of the brackets are reported with the internal containment liner in Refs. 82 and 67. By being integral with the internal containment liner, the polar crane brackets locally maintain the leak tightness barrier.

441. The RP clarified that the loads from the polar crane have been provided by the polar crane manufacturer and are based on results from a finite element model of the crane system. The polar crane is assumed to be supported by six brackets on each side of the polar crane at any one time. The reactions that have been provided include forces and moments in all directions and are applied to a separate finite element model of the polar crane bracket, shown in Figure F-6.1-5 of Ref. 67. Different parking positions and loading scenarios are considered for construction, operating and seismic conditions as described in Table T-6.1-3 of Ref. 67. From my assessment, I am content that the loading is adequately defined for the purpose of GDA and meets the intent of SAP ECE.6.

442. From my review, I note that thermal loads under both operational and accidental conditions are identified in the definition of the load combinations but are not applied in the analysis. Related to this, the RP confirmed in RQ-UKHPR1000-0858 that slotted bolt holes are included in the connection detail to accommodate the range of thermal movements anticipated under DBA loading. Although this explains why the RP has omitted thermal loads, this connection is assumed to provide lateral restraint under seismic loading and slotted holes may result in sliding and impact within the bolted connection. These aspects of the connection will need to be considered carefully as the design is further developed in the site-specific stage. I am content this is normal business.

443. The strength design of the polar crane brackets is outlined in Section 16.1 of Ref. 82 and is in accordance with AICS N690-18 for nuclear steel structures, as requirements are not covered in ACI359. I am satisfied that this code represents RGP. The RP has confirmed that the four load combinations are derived from the extensive list in the code; that the equipment reactions satisfy the rated capacity of the crane with accidental reactions; and that the seismic load is based on the DBE. The acceptance criteria for the polar crane bracket design is a Von-Mises allowable stress which includes a strength reduction factor of 1.5 for all load combinations. The results from the finite element analysis are reported in Table T-6.1-4 of Ref. 67. This shows the peak stress is within the allowable limit, with the seismic loading governing the design. I am content with the strength design for the polar crane bracket.

444. The design methodology for the concrete anchorages is outlined in Section 16.2 of Ref. 82. This is in accordance with ACI349 for nuclear concrete structures, as the requirements are not covered in AISC N690-18 or ACI359. I am satisfied that this approach represents RGP. The load combinations are reported in Section 6.2.3.2 of Ref. 67 and include load factors for construction and operating categories. The results are reported in Section 6.2.4 of Ref. 67, which show that the tensile capacity of the anchors is governing and is within the allowable limit. I am content with the strength design for these concrete anchorages.

445. In summary, for the purpose of GDA, I am satisfied with the demonstration of the analysis and design methodology for the polar crane brackets and consider that RGP is being followed. I expect the full substantiation of the polar crane bracket design to be presented in the site-specific phase.

4.6.10 Seismic Joints

446. From my assessment of the seismic joints that affect the internal containment, I identified two key areas for consideration as the shielding surrounding the fuel transfer system and the structures within the annulus between the internal and external containment structures.

447. From Figure F-1C-1 of Ref. 134, I note that lead bricks locally bridge the isolation joint between the BRX internal structures and the internal containment. The RP has confirmed that the lead bricks are required for the permanent shielding. The RP has committed to carry out further work to ascertain the impact of the bricks on the structural analysis and design. I am not clear how the conflicting requirements of the isolation joint and shielding will be harmonised. I consider that this is a minor shortfall that can be resolved as normal business in the detailed site-specific design.

448. The annulus between the internal and external containment structures is bridged by a network of stairs and walkways and fire compartment walls. Although the design of this access system is excluded from GDA scope, I note that further work will be required in the site-specific phase. The RP will need to demonstrate the adequacy of the seismic joint provision and the consistency of the configuration with the analysis and design assumptions. I am content that this is normal business.

4.6.11 Barrier Substantiation

449. The reinforced concrete barrier substantiation report for BRX (Ref. 58) identifies several scenarios which impose pipe whip loads on the internal containment and liner. I am content that these loads are being considered in the local design of the internal containment. The RP has confirmed that pipe whip restraints would be provided where necessary. The RP claims that providing additional walls to act as barriers is not possible for access reasons. Following consultation with the ONR Internal Hazards Inspector, I am content with this position. Detailed assessment commentary regarding the internal hazards considerations is within Ref. 41.

4.6.12 Beyond Design Basis

450. For the evaluation of the beyond design basis performance of the internal containment, for GDA, the RP carried out the following activities:

- a cliff edge evaluation under seismic loading, which comprises a seismic margin assessment of the internal containment (to EPRI NP-6041-SL), considering a 1.5DBE as the seismic margin earthquake. Alongside this, the RP provides a qualitative discussion on the hierarchy of failure modes for the internal containment.
- an ultimate capacity evaluation of the internal containment comprising:
  - a deterministic estimate of the capacity of the internal containment, above the design basis.
  - a deterministic assessment of the performance of the internal containment under severe accident loading.
- analysis to provide a suite of fragility curves for use in a Level 2 Probabilistic Safety Assessment for the UK HPR1000 design.

451. I am satisfied that this level of demonstration is appropriate for the purpose of GDA. Each of these activities is assessed in the following paragraphs:

<u>Cliff Edge Assessment</u>

452. For the seismic margin assessment, the RP has adopted the CDFM approach to produce a HCLPF capacity estimate for the building. This is the same approach as that applied for the BFX; the key differences for the internal containment assessment are as follows:

- The methodology for deriving seismic loads for the internal containment design means the enveloped results for the GDA soil conditions are considered in the design process. This means the results are valid for both soft and medium soil conditions.

- In addition to the seismic margin earthquake, the load combination for the internal containment includes consideration of the post-tensioning stresses and peak pressure due to a small-break LOCA, as suggested by Section 2 of EPRI NP-6041-SL.

453. From my assessment, I note that Section 2 of EPRI NP-6041-SL suggests a "small or medium LOCA". The RP acknowledged that, if the medium LOCA (or intermediate break) was adopted, the peak pressure would increase from 0.095MPa to 0.159MPa. The RP states that this load is small in comparison to the prestress load, and I concur with this statement. In view of the margins predicted, I am content that using loads corresponding to a small-break LOCA is not significant. For the site-specific phase, I expect pressure loads corresponding to a medium (intermediate) break LOCA to be adopted for the site-specific evaluation. I am content this is normal business.

454. The RP has considered that the predominant load path is 'through-tangential' (or in-plane) shear behaviour of the main cylindrical wall. The seismic margin assessment (reported in Ref. 73) considers the performance of the internal containment wall at a location slightly above the thickening of the cylindrical wall above the gusset (at the -2.6m AoD level). The assessment uses the empirical equations set out in Appendix N of EPRI NP-6041-SL. The HCLPF capacity is reported as 1.10g, much greater than the 0.45g (1.5DBE) peak ground acceleration target. From my assessment, I note that the breakdown of loads at the -2.6m level suggests the vertical membrane stresses due to LOCA are low when compared to the expected vertical stresses, based on hand calculation approximations. This implies there may be some local boundary effects at the chosen section which may not be present at a slightly higher elevation (approximately 1-2m higher). A higher vertical tensile stress would reduce the predicted HCLPF capacity. Therefore, although I am content with the calculations presented, I would expect to see further validation of the critical section chosen, as per the intent of SAP ECE.15. However, in view of the margins available, I am content this is normal business for the site-specific phase.

455. For the consideration of overturning failure, the RP provides a breakdown of vertical (or meridional) stresses on the tensile side of the cylindrical wall, (when considering push-pull of the containment). The RP has demonstrated that the wall is in net compression. I consider the calculations presented to be adequate. I note that the results presented for the liner strains demonstrate considerable margin, even under the 1.5DBE load case.

456. In summary, I am content that the seismic margin assessment has demonstrated that the internal containment has adequate margin under seismic loading, to meet the intent of SAPs EHA.7 and ECE.1 (specifically SAP paragraph 334) for the purpose of GDA.

IC Ultimate Capacity Assessment

457. The main purposes of the ultimate capacity evaluation are summarised by the RP in Section 2.2 of Ref. 74 as follows:

- To demonstrate the satisfactory performance of the internal containment under severe accident loading.
- To provide a deterministic assessment of the ultimate capacity of the internal containment.
- Development of a robust method for the ultimate capacity assessment of the internal containment. This method is then repeated in a series of analyses and used for the generation of a series of internal containment fragility curves, that are then used as inputs to the Level 2 PSA.

458. I am satisfied, for the purpose of GDA, that the intent of the RP's assessment aligns with the expectations of ONR TAG 20 (Ref. 4) and RGP for similar containments.

459. The potential failure zones are identified by the RP in Section 4.2 of Ref. 74. I am satisfied with the regions identified. I note that the regions are consistent with similar containment capacity assessments and published research e.g., NUREG CR-6906. From my review, I note one omission in the list of failure zones, and that is the shear failure of the common raft. Whilst I judge that this is unlikely to be a governing mode, I consider it worthy of consideration for very soft soil sites. Although the analysis presented at GDA has not considered this failure mode, the RP has committed in Ref. 74 to justify the performance of the common raft at the site-specific stage. This commitment is welcomed, and I consider that this is a minor shortfall that can be resolved as normal business in the detailed site-specific design.

460. The RP's analysis methodology is based on a non-linear finite element analysis representation of the internal containment; see Figure F-6-1 of Ref. 74. This analysis is undertaken using ABAQUS. ABAQUS is an established non-linear finite element code that allows the modelling of non-linear constitutive relations for concrete, post-tensioning, and reinforcement. ABAQUS can also simulate concrete-reinforcement bar interaction through constraint formulations. I am satisfied that ABAQUS is an appropriate software choice for the ultimate capacity evaluation of the internal containment, which meets the expectations of SAPs AV.1, AV.2 and AV.4.

461. From my review of the results and calculations presented in Ref. 74, I note that the RP has utilised the outputs from the ABAQUS model to assess different failure modes, along with hand calculations. These hand calculations assess shear failure in the equipment hatch and at the top of the gusset. I am satisfied that the RP's approach is in accordance with NUREG 1.216 (see clause C.1.f.(3)) and is suitable and appropriate for the ultimate capacity evaluation of the internal containment.

462. With respect to the modelling in ABAQUS, the model geometry consists of the internal containment cylindrical wall and dome, the gusset and the common raft foundation beneath the internal containment and gusset. The internal containment concrete is simulated by 8-noded solid elements, with fully integrated 4-noded shell elements, meshed into the concrete to simulate the internal containment liner. Longitudinal reinforcement layers are represented by shell elements, with the section area and reinforcement bar orientation specified in the section definition. The post-tensioning is represented by 2-noded truss elements. A constraint formulation is used to couple the post-tensioning and reinforcement bar elements to the concrete. The RP has not modelled shear reinforcement, which will result in any load transfer through shear being solely a function of the concrete shear capacity. For the consideration of the failure modes evaluated through the ABAQUS model, I consider this simplification to be conservative for GDA. During Step 4, the RP refined the ABAQUS model to include the cover for the equipment hatch, personnel access hatch and emergency hatch. The liner thickness around sleeves was refined, from a constant thickness of 6mm, to capture the 20mm thickenings around sleeves. I note that the hatch has been simulated as a continuous steel component, and the RP has implicitly ignored the presence of a bolted seal. For the ultimate capacity evaluation, this seal would be subjected to combined compression and shear due to pressure loading. Therefore, I consider that assuming a continuous load path across this seal appears to be a reasonable simplification. From my assessment, I am satisfied that the extent of the ABAQUS model, and that the structural details captured within the model, are appropriate for GDA, and that these are adequately refined to capture the failure modes that are evaluated.

463. The meshing strategy is summarised in Section 6.3 of Ref. 74. The mesh size is 1.0m x 1.0m x 0.24m, resulting in five concrete elements through the thickness. Although it is not discussed in Ref. 74, images of the mesh at the equipment hatch (see Figure F-

B-2 of Ref. 74) suggest a higher resolution in this region, with nine elements through the thickness. In Section 6.6 of Ref. 74, the RP claims that the validation work for the internal containment ANSYS model applies to the ABAQUS model. I do not fully support this claim, due to the use of non-linear material models for the modelling of the concrete and steel, in conjunction with taking the model to failure. Furthermore, I note that the mesh around the large openings, and in particular the equipment access hatch, appears different for the ABAQUS model. From my assessment, I consider that the mesh size adopted, and the additional through-thickness refinement around the equipment access hatch appears adequate. For future design phases, I expect that further mesh validation and refinement will be carried out, based on a detailed review of the results and mesh around the equipment hatch and the smaller openings. I consider that this is a minor shortfall that can be resolved as normal business in the detailed site-specific design.

464. The loads considered for the ultimate capacity evaluation are outlined in Section 7 of Ref. 74. These consist of dead loads, prestressed load, thermal load and pressure load. For the dead load and prestressed load, the method of load application is as described for the design basis M1 model. The pressure load is treated in a quasi-static manner ramping monotonically from 0 to 3.5 times the design pressure (0.42MPa). In Section 7.b of Ref. 74 the RP has discounted dynamic loading effects on the basis that this is bounded by the design peak pressure under a Severe Accident. Following discussions with the Internal Hazards Inspector, I am content with this argument. Detailed assessment commentary regarding the internal hazards considerations is available in the ONR internal hazards assessment report, see Ref. 41.

465. The thermal load applied to the ABAQUS model is a steady state temperature field, with the peak temperature taken from the severe accident temperature time curve shown in Ref. 50. The RP has considered a lower bound temperature of 5°C on the external face, and not considered any surface resistance effects, which, in reality, would reduce the predicted temperature gradient. The RP has used this modelled thermal field to calculate reduced mechanical properties at elevated temperatures. I am content that the use of a steady-state thermal field to calculated reduced mechanical properties is a conservative assumption, especially as this peak temperature of 154°C is predicted to last for 72 hours (see Figure F-7-7 of Ref. 50).

466. In summary, I am content with the loads and load combination used for the ultimate capacity evaluation and consider this in line with RGP and the intent of ECE.6 and ECE.13.

467. The boundary conditions applied to the ABAQUS model are described in Section 6.5 of Ref. 74. This states that "all degrees of freedom on the bottom surface of the containment structure are constrained and all steel materials are embedded into the concrete solid elements using embedding technique in ABAQUS without considering sliding effects". The RP claims the sensitivity study (in Section 3.8 of Ref. 69) shows that under seismic loading, very soft soil boundary conditions may increase local compressive stresses at the top of the gusset (on the outside of the internal containment) and bending moments may be underpredicted by up to 30%. I note that the sensitivity study demonstrates there is a negligible difference in results away from this region. I am satisfied with this claim and agree that, away from the common raft and gusset, this boundary condition is unlikely to have an impact on results. For GDA, I am content with this simplification. I note that the gusset liner is not the critical failure location, from the results presented in Table T-9-1 of Ref. 74. I am content the common raft foundation is unlikely to be the critical failure location. As the use of a fixed boundary condition on the entire underside of the model prevents assessment of the common raft foundation, the RP has committed to assess this failure mode further at the site-specific stage. This is also captured in paragraph 459 above.

468. During Step 4, the RP has further developed the ABAQUS model to explicitly include the equipment hatch. This has resulted in the structural behaviour in this region being more realistically captured, and this has removed any simplifications made in applying the hatch local model loads to the ABAQUS model. I welcome the RP's commitment to continue refining the ABAQUS model in the site-specific phase, to also include the personnel access hatch and emergency hatch.

469. The internal containment provides structural support to the various hatches and penetrations which pass through the internal containment cylindrical wall and into the reactor building. For the analysis of the internal containment, I note that the RP has assessed the load effects from the hatches and penetrations onto the internal containment structure. The RP also uses the ABAQUS model to derive boundary conditions for local analysis models used for the design of the equipment hatch and personnel access hatch. It appears displacements from the ABAQUS model are implemented as applied displacements in the local hatch models. I expect improvements to the RP's articulation of how boundary conditions are applied to these local models.

470. For pipe penetrations, the load from the pipe is applied to the ABAQUS model, and for the local pipe models, the internal containment is treated as a constraint. Whilst not stated explicitly, this appears to be treating the internal containment as an undeformable object for considering loading from the local pipe on the internal containment. The assessment of these local models is outside of the GDA scope, so will require more detailed assessment in the site-specific phase. Despite this, I consider that the compatibility of boundary conditions appears to be reasonable.

471. In summary, for the purpose of GDA, I am content with the boundary conditions adopted for the ultimate capacity evaluation. Further work to improve the narrative in the reporting and to assess local models more thoroughly is expected; however, I am content that this is normal business.

472. For GDA, the RP has produced results using both best estimate material properties and code minimum material properties in accordance with US NUREG 1.216. The RP's supplementary hand calculations are either deterministic or probabilistic calculations. The deterministic calculations are based on code minimum properties. The probabilistic calculations are based on mean material properties with appropriate uncertainty values modelled. I am satisfied that the RP's approach is in line with RGP.

473. For the consideration of reduced mechanical properties at elevated temperatures, material properties at temperature for the internal containment liner are based on tabulated data reported in Ref. 82. For the tendons and reinforcement, the mechanical properties at temperature are derived based on rules from BS EN 1992-1-1. Related to this, I note that the RP's assumption of a steady-state temperature profile (see paragraph 465 above) through the section will result in higher temperatures and greater reduction in material properties. I am content with this approach, which I consider to be conservative. The reduced mechanical properties at elevated temperature do not appear to have been considered for the supplementary hand calculations. For the deterministic GDA evaluation, I am content that this is of low consequence, as these failure modes are not governing the containment performance. I consider that this is a minor shortfall that can be resolved as normal business in the detailed site-specific design. Related to this, I note the RP does not appear to have adopted reduced material properties at elevated temperatures for the internal containment fragility assessment. This is inconsistent with the approach for the ultimate capacity evaluation and will need to be revisited at the site-specific stage; see paragraph 490 below.

474. For the constitutive modelling of concrete and steel components, the RP has adopted non-linear material models for all components. For the concrete modelling, a concrete

damaged plasticity model is adopted, which is capable of simulating tensile cracking and compression softening behaviour. The RP's model does consider the tensile capacity of the concrete in the constitutive model (contrary to NUREG 1.216 C.1.e). From an analysis / computational perspective, modelling zero tensile capacity is likely to be impractical and would be detrimental to the model stability and achieving reliable results. Therefore, I am content that this deviation from NUREG 1.216 is reasonable. For the liner and reinforcement, the material behaviour is assumed to have bilinear stress-strain properties. For modelling the behaviour of the tendons, a power law relationship is adopted. I am satisfied with these assumptions.

475. In summary, I judge that the material properties and constitutive modelling accords with RGP and is adequate for GDA.

476. For the assessment of failure, the RP is applying the guidance in US NUREG 1.216 to develop the acceptance criteria for the ultimate capacity evaluation. I consider the adoption of US NUREG 1.216 to be compatible with other codes used for the internal containment and represents RGP. The definitions of failure are provided in Section 5.1 of Ref. 74 and comprise structural integrity and functional integrity failure definitions. Based on these definitions of functional and structural integrity, the RP has considered functional integrity as applicable to the internal containment liner, and structural integrity as applicable to the other components that form the internal containment (concrete, reinforcement bar and tendons). I consider this distinction a reasonable approach. In reality, both functional and structural integrity have been assessed based on evaluating the strain limits of the different components of the internal containment. The functional and structural failure limits adopted by the RP are summarised in Section 8 of Ref. 74. From my assessment, I note the following:

- For local functional failure of the liner, the RP has adopted the membrane strain limit of 0.3% based on the ACI359-17 limits adopted for the design-basis assessment. NUREG 1.216 does not specify failure strains for local regions but does suggest a strain of 0.4% for areas that are away from discontinuities (which the RP has adopted for what it terms as 'integrated failure'). I consider the adoption of this strain limit to be conservative, considering the expected failure strain of P265GH steel.
- For local structural failure of the reinforcement bar, the RP has assumed a failure strain of 2.5%. The RP claims this is based on EN1992-1-1. From my review, this corresponds to the plastic strain limit for B500A for reinforcement. I note that a higher strain limit of 7.5% would be considered appropriate for the B500C reinforcement which the RP has committed to use: hence I consider this is a conservative strain limit for this analysis.
- The RP has adopted a total strain limit for the tendons that are away from discontinuity of 0.8% in accordance with NUREG 1.216. This total strain includes the initial strain induced by prestressing. I am content with this limit, but the considerations of paragraph 481 below should be noted.
- For the local limit on tendons strains, the RP has based their adopted limit of 1.0% on guidance in Chapter 9, Volume 2 of EUR. The clause from EUR that the RP refers to does not suggest the use of this tendon strain limit, but it is noted informally as a possible value. Whilst I do not consider this to be a robust basis for the strain limit, I note that this value is conservative compared to the value adopted for the UK EPR ultimate capacity evaluation; see Ref. 135. Furthermore, I note that this structural failure limit is not reached for the tendons, so I am content to accept this for the purpose of GDA.

477. Other strains adopted are in accordance with suggested limits in NUREG 1.216, and the strain limits are low in comparison to the failure strain of the liner, reinforcement bar and tendons. Furthermore, I am satisfied with how the RP is assessing compression and shear failure modes.

478. In Ref. 74, the RP has not committed to a target margin for the ultimate capacity. However, in response to RQ-UKHPR1000-0855 (Ref. 6), the RP has committed to a target margin of 2.5 for structural failure. I have assessed this in paragraph 154 above and am satisfied with this for GDA.

479. In summary, I judge that the acceptance criteria and target margin adopted for the ultimate capacity evaluation are adequate and meet the intent of ONR TAG 20 (Ref. 4).

480. From my assessment of the RP's analysis results presented in Table T-9-1 and Appendix B of Ref. 74, I note that the RP considers failure to be achieved when strain in a single element exceeds the limits defined in Section 8 of Ref 74. From my review of the results, it is clear that the critical failure region are local regions of the internal containment liner. In particular, these regions are directly adjacent to the large openings (equipment hatch, personnel hatch and emergency personnel hatch). The governing failure mode appears to be the area around the equipment access hatch. For this area, the RP in Section 10 of Ref. 74 provided hand calculations for the shear failure of the hatch and for the capacity of the stiffeners anchoring the equipment hatch sleeve into the concrete. The results indicate that the liner remains the critical failure mode for this region. I am content with this conclusion.

481. With respect to the failure of tendons, the RP's approach applies an initialising load, representative of the prestressing, ahead of the main analysis. This approach results in the initial strain not appearing in the strain output during the analysis. As a result, the analysis results do not allow the RP to directly assess tendon strains against the tendon limit of 0.8% away from discontinuities. The RP has assumed, based on NUREG 1.216 C.1.f.(2). that the strains in the tendons before pressurisation is typically 0.4%, and therefore a strain output of 0.4% from the ABAQUS analysis is considered as tendon failure. However, from my assessment I note that stress plots of the tendons suggest the initial strain in some elements appears to be up to 0.6%. The RP has subsequently provided additional spot checks on these elements with high initial strain. In general, these appear to be the vertical tendons which do not experience the same increase in strain as the pressure load is increased. The results suggest that the method for assessing the allowable tendon strain limit being reached in the ABAQUS analysis is appropriate for identifying the first failure point. However, I expect a more rigorous and automated approach to be considered by the RP at the site-specific stage to minimise the chance of error. As the pressure associated with this tendon failure mode is well above the governing failure pressure for the area around the equipment access hatch, I do not consider this significant for GDA. Therefore, this can be resolved as normal business.

482. For the associated hand calculations in Ref. 74, the RP has presented a suite of Latin Hypercube Simulations that are used to estimate the structural capacity. This approach allows for fragility functions to be developed for the shear failure modes for the equipment hatch and gusset, and for these failure modes to then be considered in the internal containment fragility assessment. For the deterministic ultimate capacity evaluation, the RP has extracted results based on a 95% exceedance probability (representative of code minimum values). Although not exactly like-for-like, I am content with the RP's approach for these calculations.

483. The results show that the liner failure at the junction with the equipment hatch is the governing failure mode (ass Figure F-B-2 of Ref. 74). I consider this reasonable considering the change in liner thickness from 20mm to 6mm in this region. Based on the failure definitions used, the analysis predicts a functional failure margin of 2.61 x design pressure and a structural failure margin of 3.40 x design pressure. This result compares positively with the original acceptance limit of 2.50 x design pressure for structural failure. Overall, I am satisfied with the analysis results presented and am content that the margins demonstrated meet the intent of SAP ECE.1 (SAP paragraph 334) and ECE.13.

484. With respect to the severe accident assessment, the RP has provided results in Ref. 74 for severe accident pressures of 0.383MPa and 0.44MPa. Table T-8-1 of Ref. 50 notes that for the assessment of the internal containment under severe accident loading, the applicable load combination has a load factor of 1.0 for the design pressure, as opposed to the 1.5 factor considered for design basis conditions. The design pressure is 0.42MPa, therefore the factored design basis pressure of 0.63MPa is well above the 0.44MPa considered for the severe accident analysis. I note that the maximum temperature for the severe accident analysis is slightly higher than that considered in the design basis condition (145°C for the design basis condition compared to 154°C for the severe accident analysis). I am content that this load is of short duration and the actual impact on the structure, if modelled more accurately, is likely to be small. In summary, by inspection, I am satisfied that the internal containment performance under a severe accident is bounded by the results at the design basis condition.

485. For the validation of the internal containment ultimate capacity evaluation, the RP has focussed on validating the ABAQUS analysis method and has presented the following:

- Hand calculations validating the standard zone capacity, presented in RQ-UKHPR1000-1488 (Ref. 6). I am satisfied that these results show good correlation with the ABAQUS analysis predictions for structural and functional capacity in the standard zone.
- In Appendix A of Ref. 74, the RP has provided a comparison of results from the ABAQUS model against the M1 model (used for design basis analysis). Results are presented for an area of the standard zone at a timestep in the ABAQUS model corresponding to the 1.5 x Pressure + Temperature. This comparison demonstrates similar strain profiles and reasonable correlation in strain magnitudes. I note that the results generally show the strain results for the ABAQUS model to be slightly lower, which appears reasonable considering the ABAQUS model also simulates the liner, tendons and reinforcement.

486. Whilst I am content that the RP has sufficiently met the intent of SAP ECE.15 for GDA, I note that the RP's validation is in places difficult to interpret. This is partially due to the number of different iterations of the ABAQUS model developed during Step 4. This has presented a challenge in understanding which model is being presented in the RP's submissions. I consider that this minor shortfall can be improved in the site-specific phase as normal business.

487. In summary, from my assessment of the internal containment ultimate capacity evaluation, I am satisfied for GDA that the overall methodology for undertaking the ultimate capacity evaluation has met RGP. Furthermore, I am content with the margins to failure predicted and consider that for GDA the intent of ONR SAPs ECE.1 and ECE.2 (SAPs paragraphs 334 and 337), EKP.1 and EKP.2 have been met.

488. I note that modifications to the ABAQUS analysis model have either been made as part of the ultimate capacity evaluation during Step 4 or have been committed to by the RP at the site-specific stage (see Table T-5-1 of Ref. 74). These modifications are:

- Inclusion of the cover for the equipment hatch, personnel access hatch and emergency hatch.
- Mesh refinements around each of these three hatch areas.
- Refinement of the liner thickness around the equipment access hatch (previously the liner was modelled as 6mm everywhere, now the 20mm thickenings are captured).
- Material properties at temperature used for assessment.

This is commendable. However, the suite of analyses undertaken to derive the internal containment fragility curves presented in Ref. 136 are based on a previous revision of the ABAQUS analysis model. I expect that in the site-specific phase these enhancements to the ABAQUS analysis methodology will be implemented in the suite of ABAQUS models used for the development of internal containment fragility curves discussed in paragraph 490 below.

Internal Containment Fragility Assessment

489. An important component of the Level 2 PSA is the development of fragility curves, which describe the probability of loss of containment as a function of the internal pressure. A separate fragility curve is developed for each failure mode, based on simulations using the ABAQUS model assessed above, supplementary models, and / or hand calculations. The fragility curves for loss of containment are documented by the RP in Ref. 136. This is a specialised technical area that I have subjected to expert review, recorded at Ref. 28. This review covered the following, some of which is recorded above:

- Methodology for assessing containment failure
- Statistical adequacy of simulation approach
- Consideration of aleatoric variability
- Assessment of epistemic uncertainty
- Development of fragility curves

490. From my assessment, I record the following points:

- As per paragraph 488 above, it is unclear whether the enhancements to the ABAQUS model and analysis will be adopted for the development of internal containment fragility curves in the site-specific phase, see AF-UKHPR1000-0231.
- For the internal containment fragility assessment, material properties are based on mean material properties, compared to code minimum values used for the deterministic ultimate capacity assessment. This is acceptable and expected for a probabilistic assessment. However, the RP does not appear to have considered reduced mechanical properties at elevated temperatures for the internal containment fragility assessment. This is inconsistent with the approach for the ultimate capacity evaluation and should be revisited at the site-specific stage, see AF-UKHPR1000-0231.
- For the consideration of aleatoric variability in the internal containment fragility assessment, the RP has simulated parameters based on a normal distribution, rather than lognormal. The RP has acknowledged this and has committed to addressing this at the site-specific stage. I am content that the effect of this error is of low consequence for the small variabilities in the simulations, see AF-UKHPR1000-0231.
- Construction and geometric uncertainties have been ignored in the internal containment fragility assessment. The difference in raw fragility values is notable when these uncertainties are considered. However, I note the dominant influence of epistemic uncertainty on the internal containment fragility results and that the sensitivity analyses carried out by the RP shows a negligible effect on the Level 2 PSA. Therefore, I consider this omission of low consequence for GDA. Nevertheless, for the site-specific phase I expect these aspects to be fully incorporated into the fragility assessment. This is captured by AF-UKHPR1000-0231.
- The RP has committed to adopting a value of 0.14 for considering epistemic (model) uncertainty in the development of internal containment fragility curves at the site-specific stage. To provide confidence at GDA, the RP has assessed sensitivity to this value and shown that for epistemic uncertainty values up to 0.20 there is minimal impact on the large release frequency. I am content with this for GDA, and I capture the site-specific commitment in AF-UKHPR1000-0231.

- The RP has committed to develop combined fragility curves to feed into the Level 2 PSA, rather than their current assumption of only taking the fragility curve based on the first failure mode. For GDA, the RP has provided confidence through sensitivity studies that large release frequency predictions are insensitive to this assumption. I am content with this provision for the purpose of GDA. I capture the requirement for further work in assessment finding AF-UKHPR1000-0231.

491. In summary, for GDA, I am satisfied with the methodology for developing internal containment fragility functions, and am content with their use as part of the Level 2 PSA. The further work outlined above for the site-specific phase, is subsumed within assessment finding AF-UKHPR1000-0231 below.

AF-UKHPR1000-0231 – The licensee shall, as part of the detailed design, demonstrate that the analysis to derive the internal containment fragility functions is consistent with the deterministic analysis for evaluating the internal containment ultimate capacity and is in accordance with relevant good practice.

4.6.13 Constructability and EIMT

492. The RP has reported on constructability and EIMT, presented in Refs. 75 and 78, which provide a high-level overview of:

- The fabrication and erection of the liner.
- The sequence of concrete pours for the internal and external containment structures. This sequence includes commentary on the temporary stability, the associated falsework and formwork, and the prestressing sequence.
- The construction personnel and equipment access within the annulus between the internal containment and external containment, specifically for construction of the external containment.
- The falsework requirement for the external containment dome construction, and the loads this might impose on the internal containment.

493. From my assessment, I consider that the two reports demonstrate an adequate consideration of the construction and conventional safety aspects at GDA. The RP has also included informative photos from the construction of similar plants in Ref. 75. I note the following positive observations:

- Protection of the exposed ends to the prestressing tendons (Figure F-3.1-7 of Ref. 75).
- Use of a purpose-made tendon jacking platform (Figure F-3.1-6 of Ref. 75).
- Liner segments being lifted with access walkways and with the edge-protections pre-installed (Figure F-3.1-4 of Ref. 75).
- Lugs cast-in to provide temporary support the pre-stressing jack (Figure F-3.1-8 of Ref. 78).

494. With respect to the construction of the external containment dome, two options are presented:

- The first has been used previously in China, using traditional temporary falsework and formwork bearing onto the internal containment (Figure F-C-1 of Ref. 75).
- The second employs a self-spanning permanent steel dome formwork with ribbed stiffeners, not dissimilar to the internal containment domed liner (Figure F-C-5 of Ref. 78).

The RP has not committed to which approach will be taken forward into the site-specific phase. I am satisfied that both options appear viable, and I am content with the RP's demonstration for the purpose of GDA. I do note that each option will create different temporary stresses on the supporting structures which will require specific consideration as part of the detailed design.

495. Many of the non-standard zones of the internal containment have limited prestress, due to the deviation of tendons. The non-standard zones therefore rely on reinforcement to resist local stresses caused by penetrations, equipment reactions and anchorages. Whilst cognisant of the GDA scope, I have assessed some sample reinforcement drawings and photographs from FCG3 to gain an appreciation of the design challenges in these regions. From my assessment, I consider that the evidence presented in GDA is adequate for the purpose of GDA and provides confidence for the site-specific phase. I specifically note the following positive points:

- I commend the RP for indicating that digital techniques will be considered in the site-specific phase, such as using 3-D digital models to mock-up areas of congested reinforcement to ensure the individual reinforcement bars and the concrete can be placed.
- The RP has indicated that they will explore the use of headed reinforcement bars to overcome the challenge of congestion. In this way, the RP has shown good awareness of the challenges of dense reinforcement, and that options (including couplers and headed bars) can alleviate congestion to some extent. I am content that this is consistent with the approach taken on other UK new nuclear build projects.

496. Although the evaluation of staged construction is omitted from the GDA scope as noted in paragraph 208 above, this is particularly important for the internal containment for the following reasons:

- The structure is prestressed and will be subject to a stressing sequence.
- The construction of the domed roof will exert and lock in forces within the walls.
- The structure may be required to provide temporary support to the external containment domed roof.
- The design is to ACI359, which imposes elastic permissible stress limits.

497. Therefore, the licensee should note that thorough development and analysis of the construction stages will need to be completed at the site-specific design stage.

498. I further note that the installation of the grout into the tendon sleeves is an important aspect of the Internal Containment construction methodology and critical to the long-term protection of the tendons. I consider this a site-specific matter that will need careful consideration and potential mock-ups and testing to ensure the design intent is achieved. I also note that RGP, for example NUREG 1.107, should be considered along with OPEX from the construction of other sites to help refine and optimise the approach for the tendon grouting and meet the intent of SAP ECE.18. I am content that this represents normal business.

499. With respect to EIMT, the instrumentation for monitoring the internal containment is outside the scope of GDA. Nevertheless, as discussed in paragraphs 409 and 410 above, I am satisfied that this monitoring system is being developed based on the RP's tendon failure analysis methodology. This will require further assessment as part of normal business in the site-specific stage.

4.6.14 Strengths

500. During my assessment recorded above, I have noted the following strengths:

- The RP has identified a suitable set of codes and standards for the design of the post-tensioned reinforced concrete structures. In general, these are internationally recognised codes of practice.
- The RP has adopted conservative seismic and static analysis processes using established and respected finite element codes that have widely acknowledged technical provenance.
- The RP has developed their methodologies by applying learning and experience from other similar internal containment designs.
- The RP has systematically defined the design parameters applicable to the design of containment reinforced concrete structures.
- The RP has defined and documented a clear methodology for the design and analysis of the internal containment structures that adheres to proven engineering practices.
- The RP has sufficiently verified and validated the methods, including analysis and design tools, and design inputs and outputs, using suitably independent methods and studies.
- The RP's approach to beyond design basis and cliff-edge for the internal containment using the EPRI, HCLPF and CDFM approaches, in combination with the deterministic and probabilistic ultimate capacity evaluations, provides confidence in the robustness of the internal containment.

4.6.15 Outcomes

501. In summary, from my assessment of the internal containment recorded above, I am satisfied for GDA that the design basis analysis for the internal containment adequately meets RGP and the intent of SAP ECE.13. I am satisfied that the beyond design basis margins established by the deterministic ultimate capacity evaluation are sufficient to satisfy SAPs ECE.1 and ECE.2 (SAP paragraphs 334 and 337) and that severe accident scenarios are appropriately considered. With respect to the probabilistic evaluation, I consider the fragility functions to be adequate for GDA purposes.

502. From my assessment, I have raised 8 assessment findings to cover address matters that require resolution as part of the site-specific or detailed design phases. As highlighted above, these are primarily associated with further refinement and validation of the modelling approach for the internal containment, the integration and use of local models, post-processing of analysis results, and improvements to the derivation of fragility functions. These are detailed in Annex 4.

503. Further to the above, I have identified several minor shortfalls and normal business items in the above sections.

4.6.16 Conclusion

504. Based on my assessment of the internal containment above, for GDA I am content with the RP's design principles and methodology and the demonstration of its application. I consider that the evidence presented is in accordance with RGP and meets the intent of the ONR SAPs. Therefore, for the internal containment sample area, I am satisfied that the RP has fulfilled the purposes of GDA.

4.7 Application of Design Principles and Methods – Sample 3 – Common Raft Foundation

505. The information presented in Annex 5 of this report (Section A.4) explains the structural form of the common raft foundation, illustrating the extent of the common raft and the buildings on and around it.

4.7.1 Design Requirements

506. There is not an individual BoSC for the common raft foundation; instead the safety case requirements are set out in the structure-specific BoSCs that the common raft supports (Refs. 31, 32, 33, 34). The BoD for the common raft (Ref. 53) presents the engineering requirements breakdown schedule, collating the information from the safety function requirement schedules from the other BoSCs. Ref. 53 links the design requirements (D1:D4, as discussed in paragraph 75 above) to the design acceptance criteria. Ref. 53 provides traceability back to the engineering requirements ID reference number in each structure-specific schedule. This traceability is useful in the absence of a BoSC for the common raft. I expect that, in the site-specific phase, as part of normal business, this schedule will be developed further to place any requirements on the quality of construction and local requirements e.g., around pits or liners, which are currently not captured. For the purposes of GDA, I am satisfied that the links within the documentation provide sufficient detail for deriving the design requirements for the common raft foundation.

4.7.2 Structural Form

507. This generic UK HPR1000 design is based on the reference design of FCG3. As part of GDA, I expect there to be stated and / or apparent logic to the design of the extents and the profile of the common raft foundation, with due consideration for constructability. As part of my assessment, I note that written justification for historical design decisions and evidence to demonstrate that the foundation concept is reducing risks in line with the ALARP principles is not presented. Furthermore, information regarding the rationale for the layout, structural form and extent of the common raft was not available e.g., achieving an approximately symmetrical form for the common raft. Nevertheless, I recognise that the design for the common raft is based on an existing mature design. I note that the reactor is approximately central to the common raft, with the buildings adjacent to the reactor building founded at a common level. The RP has inferred the benefits of this design strategy as a means to control the differential settlements between the buildings, which is important for services between buildings (Ref. 77). For the purposes of GDA, I acknowledge that the logic for aiming to design a symmetrical form is well understood; therefore, I have not pursued a line of inquiry to locate this written justification. I note that the decisions on the extent of the common raft foundation have had an impact on the design of adjacent structures, which is discussed further in Section 4.7.4 below.

508. At the start of Step 4, the thickness of the common raft foundation was based on the FCG3 design at design reference DR1.0 which, when applied to the UK generic site profile, needed to be thickened uniformly by 0.8m, captured by design reference DR2.1. Using the thicker profile, the analysis presented in Ref. 63 identifies that there are still high utilisations, with little to no margin in the design[30], with simplifications that require refinement in the site-specific stage. Therefore, I expect the GDA design will be sensitive to site specific conditions and will require further modification during detailed site-specific design. I am content this is normal business.

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[30] Reference 63, Table T-11-3 states the peak utilisations for sampled elements without averaging are 1.29 in flexure and 1.08 in shear. The RP stated that they expect averaging to reduce these utilisations to below 1.0, but this has not been demonstrated during GDA, and the results for other elements outside the sample reported herein have not been assessed.

509. Regarding constructability considerations, from my review of the reinforcement layout, I noted that the reinforcement within the common raft foundation requires a change in the orientation of the reinforcement, from an orthogonal grid layout (beneath reactor building internal structures) to cylindrical (beneath internal containment and external containment), then back to an orthogonal grid (beneath the fuel building (BFX) and safeguard buildings). The RP states this arrangement is primarily to demonstrate that there is withstand of LOCA loadings. The RP has shared Figures (extracted from drawings) and photographs to demonstrate the constructability of this reinforcement as constructed at FCG3, which demonstrates modest congestion despite the amount of reinforcement bars. The Figures shared did not illustrate the reinforcement from the prestressing gallery below, nor the shear reinforcement expected below the gusset, or the change from orthogonal to cylindrical orientation. To provide further assurance, the RP presented Ref. 75, which includes 3D modelling of the reinforcement that was used for construction of FCG3 (assumed to include these missing factors). From my assessment, I am satisfied this provides assurance that the RP understands the challenges in the reinforcement construction. The examples from FCG3 demonstrate that the common raft foundation constructed at FCG3 was not overly congested, with a similar configuration to be used for the generic UK HPR1000. Therefore, for the purposes of GDA, I am content that the design decisions associated with the structural form and geometry have not needlessly created challenges for construction.

4.7.3 Loads and Load Combinations

510. The loading on the common raft foundation is presented in the BoD (Ref. 53) and SADR (Ref. 63) as a combination of the direct loads acting on the common raft foundation, and the indirect loads which are transferred from buildings above. Reference is made to the individual BoDs for the buildings for the derivation of the indirect loads. For my assessment of the common raft foundations, I have focussed on the direct loads. For assessment of a sample of the indirect load derivation, see the BFX loading assessment in Section 4.5 above.

511. The analysis and modelling require the geotechnical properties of the generic site envelope to be characterised both statically and dynamically.

512. For direct loads, the RP has considered the following loads: dead loads, equipment loads, prestressing loads, earth pressures, normal operating and accidental temperatures, accident pressure, internal flood loads and seismic loads. Of these, I consider all the chosen loadings appropriate except for the relatively low construction live load of $4kN/m^2$, and the potential minor discrepancy in loads from equipment that have assumed an allowance for the mass of the plinths. I consider this a minor shortfall for GDA that can be resolved in the site-specific phase as normal business.

513. With respect to the load combinations, these are presented in the BoD (Ref. 53) and SADR (Ref. 63), the latter referencing the former. These reports list the combinations in the format presented in the codes (ACI349-13 and ACI359-17 Chapter CC-3230) which I am satisfied constitutes RGP. Ref. 63 provides narrative of the decomposition of the combinations for strength and serviceability design, with Appendix H of Ref. 63 giving decomposition of the ACI359 load combinations. For further information on the decomposition of the ACI349 load combinations, see Section 4.5 above. I consider this to be a comprehensive set of code-based load combinations for the general areas and the containment of BRX that, for the purposes of GDA, meets the expectations of ECE.6.

4.7.4 Static Geotechnical Analyses and Global Stability

514. The methodologies for the common raft foundation with respect to the overall stability analysis, and computation of settlements and tilts are unique and require static and dynamic SSI to be considered. The geometry directly influences the settlements and

inclinations that are a critical input for the design of SSCs[31]. Furthermore, the SFRs for the design basis, beyond design basis and severe accident LoCA conditions vary with location based on the function of the individual facilities it supports.

515. The analysis and modelling for the common raft foundation require the geotechnical properties of the generic site envelope to be characterised both statically and dynamically. As part of my assessment of this area, I raised and closed Regulatory Observation RO-UKHPR1000-0009 (RO-0009), (Ref. 7). My assessment of the RP's response to the three RO actions is summarised below. For the full details of this topic, Ref. 101 should be referred to.

516. For RO-0009 Action 1, I assessed whether the RP has justified and defined a consistent set of dynamic and static geotechnical parameters that adequately represent the GDA envelope with respect to the UK generic site. This covered the following topics:

- Generic site shear wave velocity envelope definition
- Generic site allowable bearing pressure definition
- Generic site ground stiffness definition
- Compatibility of representative *'Target Site'*[32] stiffness with generic shear wave velocities
- Compatibility of generic bearing pressure presented in respective documents
- Groundwater Level

517. Overall, from my assessment recorded at Ref. 101, I am satisfied the RP has justified and defined a consistent set of dynamic and static geotechnical parameters that adequately represent the GDA envelope with respect to the UK generic site as per the intent of SAPs ECE.7 and ECE.13.

518. For RO-0009 Action 2, I assessed whether the RP has developed and articulated adequate analysis and design methodologies for the application of the geotechnical GDA envelope to demonstrate the adequacy of the overall design concept. This covered the following topics:

- Performance limits
- Global stability
- Static settlement analyses for equipment and structural checks

519. Overall, from my assessment recorded at Ref. 101, I am content that the RP has developed and articulated adequate analysis and design methodologies for the application of the geotechnical GDA envelope to demonstrate the adequacy of the overall design concept. Furthermore, adoption of cautious ground stiffness properties is considered to provide margin in the GDA analyses, which will aid refinement of common raft foundation design and settlement solutions at the site-specific stage, in line with the expectation of SAP ECE.13.

520. For RO-0009 Action 3, I assessed the substantiation of the UK HPR1000 generic design for the geotechnical generic design envelope, and judge whether it is deployable given appropriate levels of site-specific design optimisation. From my assessment recorded at Ref. 101, I am content with the substantiation of the UK HPR1000 generic design for the geotechnical GDA generic design envelope (see paragraph 177). I consider that this meets the intent of GDA and the expectations of SAPs ECE.7 and 13. Furthermore, based on the credible solutions outlined by the RP,

[31] *Although the GDA process does not consider site-specific geotechnical parameters, the results using the generic site envelope enable a judgment to be made regarding the foundation concept, and whether it is suitable to be progressed and optimised further in the site-specific phase.*
[32] *The 'Target Site' conditions for this GDA are based on the Bradwell B site. This site consists of approximately 80-90m of London Clay overlying Chalk, so is regarded as a 'Very Soft' site with a mean V_s30 of approximately 250-300m/s. Therefore, a focus during GDA has been on ensuring the GDA generic site envelope bounds this 'Very Soft' soil profile.*

I judge that the design is deployable given appropriate levels of site-specific design optimisation as will be expected to meet the intent of SAP ECE.5.

521. During the assessment of RO-0009, I identified the 'Residual Matters', (wording from Ref. 101 noted below in italics), with the subsequent conclusion thereafter (that which has occurred since the closure of RO-0009, Ref. 101):

- *Within GDA the RP is expected to address the inconsistency between (higher) values of static springs for 'Very Soft' (*V_s of *150m/s) site presented in T-5-4 of Ref.* 26 *compared to the (lower) values presented in T-6-6 of Ref.* 53 *and in T-B.3-2 of Ref.* 42.
  - I can confirm that the inconsistencies have been addressed with the appropriate changes incorporated in Refs. 42 and 26.
- *Within GDA the RP is expected to address the inconsistency in Refs.* 43 *and* 42 *with respect to the 'proposed generic site envelope'. Currently the bearing pressure in Ref.* 43 *is shown to be compatible with the range of generic allowable pressures derived from the generic shear wave velocities presented in Ref.* 42. *However, as the magnitudes differ it is inconsistent to present both as "generic".*
  - I can confirm that the term 'generic' has been substituted with the term 'presumed allowable' when referring to pressures in Ref. 42, which suitably overcomes this ambiguity. GDA evaluates bearing demand, and site-specific allowable pressures and bearing capacity will be defined through thorough ground investigation and assessed at the site-specific phase as part of normal business.
- *Post GDA the RP is expected to demonstrate stability against sliding via passive and / or ground improvement options.*
  - The RP has included the calculations and associated factor of safety for the undrained and drained shear resistance to Ref. 63. I am content that this has adequately demonstrated the methodology for GDA, and the results confirm the need for further design development of the common raft. This future work should demonstrate global stability of the common raft design, notably for the following:
    - Settlement and tilts predicted by the static soil-structure interaction analysis.
    - Dynamic bearing capacity under earthquake loading.
    - Sliding under earthquake loading when considering the drained and undrained shear resistance of the soil, including how the global sliding resistance impacts the design of the prestressing gallery.
    - Pounding between adjacent foundations.
  - For GDA, I am satisfied with the RP's position to differ these aspects as they are heavily dependent on the future site-specific ground characterisation and foundation design and optimisation. Nonetheless due to their significance I record this in assessment finding AF-UKHPR1000-0232 below.

AF-UKHPR1000-0232 – The licensee shall, as part of the site-specific design, optimise the design of the common raft foundation to satisfy the global stability requirements.

- *Post GDA the RP is expected to justify as appropriate the statement that 30% of the variable (live) load will be considered in assessing settlements.*

  - Whilst I am content that this percentage appears reasonable for GDA, an appropriate allowance for variables33 should be incorporated into checks at the site-specific phase (e.g., for 'very soft' sites, Vs of 150 m/s). See paragraph 522 below.
- *Within GDA the RP is expected to amend Ref.* 63 *to make clear that the omission of buoyancy in the structural checks is the cause of the difference between structural and geotechnical settlements and to confirm that omission of buoyancy is conservative.*
  - As already noted in Section 4.7.3 above, the settlement analyses for structural checks (as opposed to global stability checks) presented in Ref. 63 omit buoyancy force on the raft, leading to a discrepancy between settlements from the structural analysis presented in Ref. 63 and settlements from the geotechnical model presented in Ref. 76. I can confirm that the RP has updated Ref. 63 Section 8.4.1 and Ref. 76 Section 5.7 to better report this approach and to confirm that the structural checks remain conservative.
- *The RP is also expected to amend Ref.* 76 *and* 77 *to clarify that rigid common raft behaviour could increase differential settlements to 150mm and to demonstrate that credible solutions can accommodate this magnitude.*
  - The RP confirmed that rigid raft behaviour could increase maximum predicted differential settlements between the common raft foundation and adjacent buildings from 45mm to 150mm, thus surpassing the SSC limit of 100mm justified in Ref. 77. The RP has presented calculations confirming that a more extensive application of the three credible solutions (construction phasing, replacing ground and increased separation) in combination can still achieve the 100mm differential settlement limit, for which I can confirm the RP has appropriately updated Ref. 77 and Ref. 76. The calculations will be refined at the site-specific phase.
- *Within GDA the RP is expected to include in Ref.* 84 *all the future commitments relevant to the Common Raft, including assessment of bearing capacity and seismic sliding resistance.*
  - I can confirm that the common raft foundation DSR (Ref. 84) includes the future commitments related to the residual matters noted above, including the variable loads, the seismic sliding checks, and a site-specific commitment to complete a thorough assessment of the effects on the design of concrete creep, as well as shrinkage, early thermal cracking and detailed specification of concrete mix.

522. From my GDA assessment, I note that for the detailed site-specific phase I expect the licensee to refine their soil-structure interaction analysis methodology to account for the following:

- The simplified GDA method using a single uniform soil spring stiffness applied to each building or building zone should be refined and edge effects accounted for.
- For a soft site, the soil non-linearity and variation in stiffness with strain should be characterised and modelled appropriately. This should include validation of the proposed analysis methodology via back-analyses of testing and / or site trials.
- An appropriate allowance for variable loads should be incorporated.
- Settlement predictions should consider both short- and long-term concrete stiffnesses, thereby accounting for variation in concrete mechanical properties due to creep, shrinkage or construction staging.

[33] *RP has stated that they estimate 30% of live load likely to be sufficiently permanent to contribute to settlement, resulting in only a 3% increase in load causing settlement.*

I am content these items represent normal business.

523. In summary, for the purposes of GDA, I am satisfied that my assessment for RO-0009 recorded at Ref. 101 has confirmed the adequacy of the RP's geotechnical parameters, methodology for static SSI and global stability, and the substantiation of the GDA design for the geotechnical generic design envelope.

4.7.5 Reinforced Concrete Analyses and Design

524. The common raft foundation design (including the prestressed gallery) employs the standard design methods described in Section 4.4 above. Other aspects of the common raft foundation are discussed earlier in this report, e.g., in Section 4.4 where the global models for static and seismic analysis include the common raft foundation, and Section 4.6, where the common raft section under the BRX is analysed as part of the internal containment analysis.

525. For the common raft foundation, I have assessed the adequacy of the use of global and local models for substantiation of non-uniform regions. I assess this alongside the adequacy of the data handling, post processing and reporting of analysis output and design results, the methodology for which is outlined in Section 4.4.8. By discussing model accuracy and mesh sensitivity in earlier sections, I now focus on two geometrical attributes specific to the common raft foundation, namely local pits and the change in thickness of the raft at the perimeter of the BRX.

526. The RP presents two types of local pits; 'Type 1' are sufficiently deep to create a step in the underside profile of the raft, and 'Type 2' which are shallower and do not impact the underside profile. The RP has indicated 'Type 1' pits are included in the ANSYS analysis models, stating that 'Type 2' pits are part of detailed design therefore are outside the scope of GDA. From my assessment, I am content with this approach for the purposes of GDA: however, I note that the utilisations around some of the Type 2 pits are high. I consider that this could be significant to leak tightness checks regarding through-thickness cracking for reduced slab thicknesses in areas of high tensile stress. This point is also relevant to the M1-Gusset model, specifically the area of the gusset around the change in the thickness. From my review, I identified that the pits immediately adjacent to the gusset are not included in the modelling. As per the intent of SAP AV.1, I expect this to be an area of focus for future design development and record this in assessment finding AF-UKHPR1000-0233 below.

AF-UKHPR1000-0233 – The licensee shall, as part of the site-specific design, substantiate the design of the common raft foundation to ascertain the impact of the detailed geometrical configuration on the areas of the raft with high utilisation. This should include, but not be limited to, the pits adjacent to the internal containment gusset area.

527. I further highlight the challenge of reinforcement detailing in these local regions. For the purposes of GDA, I note that the design decisions made in GDA do not preclude the option to thicken the common raft foundation locally to the pit locations to address this, if appropriate, as part of detailed design. Further, the modelling geometry for the common raft foundation does not reflect the changes to the geometry of BFX, which is implicitly captured by assessment findings AF-UKHPR1000-0214 and AF-UKHPR1000-0223 above.

528. Regarding the modelling data handling and post processing, Ref. 63 uses contour plots as part of the presentation of the results of the post-processing, and these indicate that the RP has reinforcement results for the whole structure. For the purposes of GDA, Ref. 63 demonstrates an adequate means of presenting and reporting the results of the post processing that I consider meets the expectations of SAP AV.5.

529. The common raft foundation BoD (Ref. 53), SADR (Ref. 63) and DSR (Ref. 84) include details regarding the design life, concrete grade, concrete cover and exposure class. From my assessment, I am satisfied that the defined parameters are appropriate. I note that there is a potential for misunderstanding as a result of simplification in Table T-3.3-1 of Ref 42, where the exposure class for common raft foundation is simplified as XS3. The exposure classes are more accurately reported in the common raft foundation BoD (Ref. 53) as being XS1 (50mm cover) for internal surfaces and XS3 (65mm cover) for external surfaces. I note that these cover values will be refined in the site-specific phase. From my assessment, I note that the concrete cover stated in GDA submissions are higher than the minimum values stated in BS8500-1 (see also paragraph 179, above). It is not clear in the documentation whether these concrete cover values include an allowance for tolerance, which is important for the common raft foundation which will be cast against uneven surfaces such as the ground or blinding (usually greater than the 5mm potentially proposed by the RP). Concrete cover values are dependent on site-specific information regarding understanding of the substrate to the common raft and decisions around concrete mix type. I expect the licensee to make a clear statement regarding the adequacy of the concrete cover, including assumed construction tolerances (ΔC_{dev} to [EN1992-1-1]) once the concrete mix and raft substrate are confirmed. This will be reassessed as normal business during the site-specific phase.

530. Further associated with concrete cover values, the RP does not appear to make a safety claim on the external waterproofing membrane for the common raft. This means the exposure class decision (XS3) ignores the presence of the membrane between the concrete and the substrate. I consider this is an appropriate decision in accord with SAP ECE.16 and ECE.10, considering the membrane will not be maintainable and products currently available on the market are not guaranteed for a 100-year design life.

531. In summary, for the purposes of GDA, I am content with the reinforced concrete analysis and design for the common raft foundation. I expect further refinement to be carried out in the detailed site-specific design phase as highlighted by AF-UKHPR1000-0233.

4.7.6 Interfaces and Construction Details

532. As described in Section A.1 of Annex 5, the structures on the common raft abut each other and therefore I expect the design requirements for building interfaces to be suitably mature, demonstrably viable with consideration of buildability, maintainability and watertightness. Furthermore, the design incorporates stepped sections across the common raft foundation profile, between the different foundations for the buildings. These steps have filler board joints e.g., with BSB, which are constructed to be sufficiently compressible to avoid force transfer. Regarding the compressibility of joints between structures, the RP has stated there are relatively small differential movements of the common raft foundation during a seismic event, albeit that these calculations regarding the pounding risk at the raft interfaces, or the shear failure of the soil have not been presented at GDA. From my assessment, I am content with the RP's approach, and am satisfied that these aspects can be considered further at the site-specific phase, as site-specific inputs may result in larger movements to be accommodated by the filler board joints than the movements currently predicted. This requirement for further work is captured in assessment finding AF-UKHPR1000-0232 above.

533. The waterproof solution for the building-to-building joints is presented in Ref. 71. I note that this will need to accommodate the substantial vertical differential settlement limits that are predicted in Ref. 77. I am content that this can be resolved as normal business in the detailed design phase once the predicted differential settlements become more refined.

534. The prestressing gallery is a structure that is integral with the common raft foundation, as described in Annex 3. With regard to the sliding checks, as discussed in paragraph 235 above, the RP's methodology does not appear to consider the dynamic soil pressures. The RP does not explicitly consider the concentrated forces on structural protrusions that will result in structures, such as the prestressing gallery, acting as shear keys. Although I am content that there are options within detailed design that could address this, as noted in Ref. 28, I consider it appropriate to capture this in assessment finding AF-UKHPR1000-0232 above.

4.7.7 Constructability and Conventional Safety

535. Constructability is not explicitly discussed in Ref. 63; instead the report presents the benefits of thickening the raft, with Ref. 89 stating that the increased thickness demonstrates "good constructability". Although the RP has not presented the reinforcement detailing for GDA, I concur with the RP's statement in so far as the RP has considered reinforcement congestion by increasing the thickness of the raft in preference to increasing reinforcement density. The RP presents a reinforcement bar arrangement that typically has a 200mm pitch, which I consider adequate as this is generally regarded as ideal spacing to balance congestion and crack control.

536. As discussed in Section 4.7.2, the RP shared photographs that demonstrate the constructability of similar projects. This is further demonstrated with the information presented in Ref. 75, where the RP presents a view of a 3D model used to coordinate the placement of the reinforcement at FCG3, alongside discussion around reinforcement fixing, congestion and concrete placing.

537. Regarding the waterproofing, the RP provides discussion on protecting the external waterproofing membrane during construction in Ref. 71. This is welcomed as a recognition of good practice for the membrane to preserve the structure, even when there is no claim placed on the membrane.

538. The RP has not presented a design risk register for the common raft construction. I would expect this to be in place before the site-specific analysis and design checks are completed for the construction sequence, as some risks could be mitigated through design. I consider this to be part of normal business, as demonstration that risks are mitigated during the design stage where possible, and where residual risks remain, these are adequately and effectively communicated for future risk management.

539. I consider that the above examples provide an adequate demonstration that constructability has been considered in the RPs safety case submissions and that the construction of the common raft foundation is credible. Furthermore, I have not identified any unusual constructability challenges that have been unnecessarily included as a result of design decisions, or that conventional health and safety risks have been exacerbated by inappropriate design decisions.

4.7.8 Strengths

540. During my assessment recorded above, I have noted the following strengths:

- The design for the foundation profile was thickened during GDA to allow for the effects of the generic site envelope.
- The use of contour plots show that the RP has means to present the reinforcement results graphically, associating the results with the 3D structure.
- The static SSI and the associated credible solutions demonstrate that the GDA foundation design can be optimised as appropriate for site-specific design.
- The foundation design adequately addresses concrete durability.

- The RP demonstrated an adequate understanding of the associated construction risks that will need to be considered further in the site-specific design.

4.7.9 Outcomes

541. In summary, from my assessment of the common raft foundation recorded above, I note that RO-UKHPR1000-0009 (Ref. 101) considering the design geotechnical parameters has been raised and closed as part of GDA. Moreover, I am content that the RP has presented a thorough demonstration of the application of their design principles and methodology that is sufficient for GDA purposes. Due to the site-specific nature of the common raft foundation design, combined with the very soft characteristics of the generic site envelope, further development and optimisation work will be needed as part of site-specific design to fully meet the expectations of SAPs ECE.7 and ECE.16. Nonetheless, based on the credible solutions outlined by the RP, I judge that the design is deployable given appropriate levels of design optimisation.

542. From my assessment, I have raised 2 assessment findings to address matters that require resolution as part of the site-specific or detailed design phases. As highlighted above, these are associated with optimising the design of the common raft foundation to satisfy global stability requirements and refining the modelling and analysis to adequately capture the detailed geometry. These are detailed in Annex 4.

543. Further to the above, I have identified several minor shortfalls and normal business items in the above sections.

4.7.10 Conclusion

544. Based on my assessment of the common raft foundation above, for GDA I am content with the RP's demonstration of their design principles and methodologies. I consider that the evidence presented is in accordance with RGP and meets the intent of the ONR SAPs. Therefore, for the common raft foundation sample area, I am satisfied that the RP's demonstration has fulfilled the purposes of GDA.

4.8 Application of Design Principles and Methods – Sample 4 – BNX and BDB/BDV (SSE1 Structures on Individual Rafts)

545. The information presented in Annex 5 of this report describes the site arrangement (Section A.1), illustrating the locations of the BNX (nuclear auxiliary building) and BDB/BDV (emergency diesel generator buildings).

546. The nuclear auxiliary building (BNX) is a non-symmetrical structure situated immediately adjacent to the common raft and was selected to be sampled to assess this aspect of the RP's design and analysis methodology. The BNX is split into two zones which are structurally isolated from one another with independent foundations. BNX-I measures 43m x 50m on plan, 31m above ground with an excavation depth of -12.75m which is approximately 1m below the adjacent common raft excavation depth. BNX-II measures 32m x 21m on plan and extends 20m above ground and with an excavation depth of -9.25m, which is approximately 2.5m above the common raft excavation depth. As these buildings are immediately adjacent to the common raft, they are susceptible to the effects of structure-soil-structure interaction (SSSI), and this has not been otherwise presented in GDA, see assessment finding AF-UKHPR1000-0217. The structural analysis of the BNX was limited to the zone one (BNX-I) and assessed for the very soft (V_s of 150m/s) soil profile, seismic load case combinations and typical elements (Ref. 12). The BNX has the same seismic categorisation given to buildings on the common raft foundation[34]. Whilst the full safety case submissions

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[34] *BNX and BDB/BDV have the functional categorisation FC3, functional classification F-SC3 and the same seismic categorisation SSE-1 as do the structures on the common raft assessed earlier herein.*

have been received for the BNX, my assessment sample has focussed on the specific features that differ from the other buildings on the common raft foundation within the BFX sample area. This section of the report therefore focusses on the seismic analysis methodology for BNX, including soil-structure interaction (SSI) and SSSI effects, and the consideration of external explosion loading, which is expected to be of greater significance for BNX than other structures that are already designed to protect against malicious aircraft impact loading.

547. The two emergency diesel generator buildings (BDB/BDV and BDU/BDC/BDA) are spatially separated from the common raft foundation: the precise location of these two sets of buildings has not been decided at GDA. The RP states that they are to be sufficiently separated, designed such that one building remains functional at all times. The joined BDB/BDV building was sampled to look specifically at the application of the RPs seismic analysis methodology when considering embedment effects, which have not been otherwise presented at GDA, see assessment finding AF-UKHPR1000-0217. These buildings are embedded and have higher depth-to-equivalent-radius ratios[35], compared to the other Class 1 structures off the common raft. The BDB/BDV building is approximately 29m long, 26m wide and extends 25m above ground, to the lowest floor level of -11.3m. The RP has considered embedment effects in a sensitivity study performed for the BDB/BDV building. The diesel generators are located at ground level, with main oil storage tanks and fuel delivery pumps at the lower levels, and auxiliary systems including Heating, Ventilation and Air Conditioning (HVAC) systems on the other above ground floors. The stored hydrocarbon fuels present a heightened fire load, different to the other sampled structures within GDA. This section of the report therefore focusses on the design methodology to address this hazard within the context of BDB/BDV, alongside the sampling for embedment effects. The RP has provided the BoSC and BoD (Refs. 35, 45) for the emergency diesel generator buildings which include the high-level Safety Functional Requirements schedule.

548. Both BNX and BDB/BDV are cellular reinforced concrete structures (similar to BFX) with orthogonal external and internal subdividing walls providing the primary load paths for vertical and horizontal actions.

4.8.1 Soil-Structure Interaction (SSI) of the BNX

549. Soil-structure interaction (SSI) is an important consideration in dynamic response of a building for seismic analysis, as it can affect seismic demand and deformation of the structure. In my assessment, I sampled the RP submissions which evaluate the results to understand seismic behaviour alongside the associated validation.

550. The analysis of seismic behaviour for BNX-I is presented in Ref. 61, which has demonstrated that SSI effects are significant for the buildings off the common raft foundation, by analysing the dynamic behaviour of the building, based on a very soft (V_s of 150m/s) soil profile. As discussed in para. 213 above, in using only the soft soil profile, the seismic demand presented does not represent the full GDA site envelope. The RP acknowledges this requirement for further work in the BNX DSR (Ref. 87). The results indicate low utilisations for the internal shear walls, so the increased demand for the medium soil profile may have limited impact on the design of the BNX-I building, but Ref. 65 presents limited information on the strength design. I am content that this is a minor shortfall that is not significant for the GDA demonstration that can be resolved as normal business in the site-specific phase.

551. The analysis shows that the SSI effects vary with height, and how this impacts the supporting SSCs, based on their location within the building. For the site-specific phase, I expect the floor response spectra to envelope the peak broadened response

[35] *This ratio is used to determine whether embedment effects should be considered in the analysis, see ASCE 4-16 for further information.*

for best estimate, upper and lower bound properties of the soil conditions, as part of normal business.

552. The analysis of the acceleration response of the BNX-I foundation is presented a lower zero period acceleration (ZPA) compared to the EUR target spectra. The RP stated this is due to the relatively high amount of composite damping in the SSI system including radiation damping in the soil, which I consider reasonable, given that radiation damping is typically much higher than the 5% of the target response spectra.

553. The seismic analysis was validated with a lumped mass stick model and this is presented by the RP in Ref. 61. This study confirms how the SSI effects impact the frequencies generated with the soft soil profile. From my assessment I am content with the RP's validation and note that the level of detail presented for BNX-I exceeded my expectations.

4.8.2 Structure-Soil-Structure Interaction (SSSI) for BNX

554. As part of my assessment, I sampled the sensitivity studies of the seismic analysis for the BNX-I to confirm the adequacy of the inclusion of SSSI effects, as the BNX-I is located away from the common raft foundation. SSSI analysis will be required at the site-specific phase, and I expect the methodology to be outlined within GDA scope.

555. The RP used a full ACS SASSI 3D finite element model to analyse the SSSI effects on BNX-I; this is illustrated in Figure F-4-57 of Ref. 68. The RP's model consisted of the structures on the common raft (ANSYS Model 1) and the BNX-I building, which was referred to as 'the coupled model'. I confirm this approach is consistent with the rigorous method in ASCE4-16, which requires all structures to be included in the same model and this is adequate for GDA. Whilst the methodology used for the sensitivity study described in Section 4.8 of Ref. 68 was in accordance with RGP during GDA, I expect this to be given detailed consideration and the methodology articulated in greater detail for SSSI for all buildings off the common raft foundation at the site-specific phase. This is discussed in para. 225 above (see AF-UKHPR1000-0217).

556. Ref. 68 presents the floor response spectra for 'the coupled model' which has an additional peak compared to the individual BNX-I model. This demonstrates the impact SSSI can have on the seismic demand for the supported SSCs within the BNX-I building. Ref. 68 concludes that the SSSI effects have a significant impact on the seismic demand for the buildings that are located away from the common raft, and the structures systems and components (SSCs) therein. The RP does not provide specific comment on the floor response spectra, but I expect this to be considered in the further work at the site-specific phase. Ref. 68 concludes that the design at GDA is potentially non-conservative, but the RP does not quantify the impact. This is discussed generally in para. 227 above, and this future work requirement is captured in assessment finding AF-UKHPR1000-0217.

557. The RP provides forward commitments in Ref. 24 to analyse SSSI effects for all buildings off the common raft at the site-specific phase. It is possible that the cumulative effect of including SSSI on a stiffer soil may be more significant. The RP presents limited information on the strength design; however, I am content that this is a minor shortfall that is not significant for the GDA demonstration and that it can be resolved as normal business in the site-specific phase.

4.8.3 Seismic Joints for the BNX

558. As part of my assessment, I sampled the seismic joints for the BNX to confirm the adequacy of the seismic joint provision to accommodate longer-term static displacements due to ground settlement and tilt, as well as the seismic displacements. The seismic joints between the structures within the GDA scope are described in para. A.1.3 in Annex 5.

559. The general approach to seismic joints is discussed in paragraph 241 above. For BNX-I, the RP presents the seismic displacements in Ref. 61, and the RP's assessment of the seismic joints is presented in Appendix A of Ref. 11 as a sample calculation. The RP has calculated 171mm as the minimum separation between the BNX-I and the buildings on the common raft foundation. This was calculated considering both seismic displacement and horizontal deformation with a credible foundation solution, for which a 200mm seismic joint width would be sufficient. The RP's calculation has not included foundation displacement as required by the recently revised standard ASCE43-19. The RP confirmed this additional displacement would be approximately 14mm for the very soft (V_s of 150m/s) soil profile, which could be accommodated by a 200mm seismic joint. The RP has committed to include a sensitivity study of the SSSI effects in the seismic joint assessment at the site-specific phase. From my assessment, I consider that the foundation and soil displacements should be included in the assessment of the seismic joints at the site-specific phase in accordance with ASCE43-19. However as this is not expected to exceed the capability of a seismic joint, I consider this a minor shortfall that can be resolved as normal business.

560. The methodology for the seismic analysis sensitivity studies focussing on damping and concrete stiffness are covered in paras. 224-226 above. I note that the RP did not consider the effects on seismic displacements. Although this aspect was sampled for the BFX building, captured in paragraph 331, the results are expected to be similar for the seismic joints of the BNX. The RP acknowledged that the influence of structural damping on the seismic displacements was greater for stiffer soils, and that cracked section properties are significant for the medium (1500m/s) soil profile. The RP has committed to using the stiffness that corresponds to the stress state of the structure at the site-specific phase. It is therefore possible that the seismic displacements could increase at the site-specific phase, and that this increase could be significant for stiffer soils. I am content that this can be regarded as normal business.

561. From my assessment I note that the foundation level is different for BNX-I and BNX-II, which leads to the risk of out-of-plane forces in the structures in the event of pounding. This is discussed in the context of BNX–common raft foundation joint in paragraph 532 above and applies to the BNX-I – BNX-II joint. Furthermore, the points made in paragraph 533 apply to this example of the BNX-I – BNX-II joint with respect to waterproofing solutions and the anticipated movement. I am content that these matters can be resolved as normal business for these structures.

4.8.4 External Explosion for the BNX

562. The BNX is not designed to withstand the malicious aircraft impact and so I have sampled the impact of external explosion loading for BNX-I in my assessment. Ref. 42 outlines the pressure and duration of the blast wave, as the loading is applied as an equivalent static pressure to the external walls of the building. Section 7.15.1 of the BNX SADR (Ref. 65) describes the various factors applied to the pressure loading to account for dynamic amplification, reflection or focussing effects. The RP confirmed this approach is in accordance with ETC-C. From my assessment, I consider this approach to be over-simplified and I expect the assumptions to be reviewed at the site-specific phase, because for some elements of BNX-I, the external explosion is the governing load case which will influence the design. As the external explosion loading is not expected to be a significant hazard for the civil engineering design, I confirm that

the simplified approach adopted is sufficient for the purpose of GDA, as the detailed design phase can be adapted to accommodate the loads upon refinement.

563. The BNX SADR (Ref. 65) differs from the SADRs for structures on the common raft where it states that global stability checks should be undertaken for external hazard loading. The seismic loading was found to govern the BNX-I stability checks, but I note that external explosion may be more significant for more slender buildings that are not designed to withstand malicious aircraft impact. I consider that these checks should form part of the SADR methodology for other buildings but am content this can be resolved as normal business.

4.8.5 Fire Design for BDB/BDV

564. For my assessment I expected the RP to provide an analysis and design methodology for quantifying hydrocarbon fire loads acting on the civil engineering structure, with assessment criteria for evaluating the performance of the affected structure(s).

565. The BDB/BDV building stores hydrocarbon fuel, which has the potential to raise the fire load to a ‘non-standard’ fire load, rendering the standard fire design method inappropriate. The standard fire design method was sampled for BFX, see para. 304 above. The fire design methodologies are generally presented in the individual BoSC and BoD reports, rather than the overall SADMS (Ref. 25), which means that, whilst the information in Ref. 25 is applicable to BDB/BDV, it does not provide sufficient detail.

566. The scope for GDA included the diesel generator buildings BoSC (Ref. 35) and BoD (Ref. 45) with a high-level safety functional requirements (SFR) schedule or engineering requirement breakdown schedule. These schedules are used to supplement the project SAMS (Ref. 24) and the SADMS (Ref. 25), as demonstrated in the BFX building assessment (see para. 274 above).

567. The SFR schedule for the BDB/BDV building presents two safety functions broken down into six engineering requirements[36], without further decomposition for the structures, systems and components. The design methods in the BoSC (Ref. 35) and BoD (Ref. 45) make reference to design codes and standards which are consistent with the SADMS (Ref. 25), which provide assurance that the generic design aspects can and will be substantiated.

568. The RP uses numerical simulations to determine the fire load and has presented a single fire load from a room in the BDA building as representative of a hydrocarbon fire. This fire load exceeds the ISO 834 fire resistance ‘standard fire curve’. The RP has not provided evidence to confirm this fire load is bounding for the BDB/BDV.

569. There are no details on the hydrocarbon fire design methods or the fire performance requirements for the BDB/BDV building. The RP has not defined the hydrocarbon fire loads for the BDB/BDV in their submissions. The BoD (Ref. 45) lists that the reinforcement covers are defined in accordance with the durability requirements, with a 30mm concrete cover specified. I consider that a 30mm cover could be inadequate for the fire design, depending on the fire load and resistance period.

570. The BoD (Ref. 45) states that the internal hazard assessments of the buildings outside the common raft foundation are yet to be completed. Therefore, further work is required to confirm the fire loads and the compartments they apply to, formally documenting this in the safety case submissions.

571. The RP claims that even if divisional barriers were lost, there would be sufficient redundancy due to the physical separation of the BDB/BDV and the BDA/BDC/BDU

[36] This is comparable to the 100 engineering requirements presented in the equivalent full SFR schedule in Ref. 35 for the BFX.

buildings. I consider this is a reasonable argument; however, the hydrocarbon fire load is not bound by the ISO834 standard fire curve. Therefore, I expect a methodology to be in place, alongside structural performance requirements, to demonstrate that the non-standard fire case can be substantiated. This further work is captured by assessment finding AF-UKHPR1000-0234 below.

AF-UKHPR1000-0234 – The licensee shall, as part of the detailed design, justify the methodology for the substantiation of the claimed barriers against hydrocarbon fires.

4.8.6 Strengths

572. During my assessment recorded above, I have noted the following strengths:

- The level of detail presented for the validation of the BNX-I seismic analysis exceeded my expectation.
- The sensitivity study for the SSSI effects used the rigorous method that is compliant with ASCE4-16, modelling all the structures on the common raft and the BNX-I within a single 3D model in ACS SASSI.

4.8.7 Outcomes

573. In summary, from my assessment of the SSI and SSSI for the BNX recorded above, I have gained assurance in the RP’s methodologies for structures of this type and classification. Subject to a more complete analysis methodology being developed, I consider the approach appropriate for the site-specific design and in accordance with RGP and the intent of SAP ECE.12 and ECE.14. For the BDB/BDV buildings, I consider that further work is required in detailed design to ensure an appropriate methodology for non-standard fires is established that meets the intent of SAP ECE.13.

574. From my assessment, I have raised an assessment finding regarding the methodology for the substantiation of barriers against hydrocarbon fires. This is detailed in Annex 4.

575. Further to the above, I have identified several minor shortfalls and normal business items in the above sections.

4.8.8 Conclusion

576. Based on my assessment of the of the SSI and SSSI for the BNX building above, for GDA I am satisfied with the differences in design methodologies for SSE1 structures not on the common raft foundation. I consider that the evidence presented is in accordance with RGP and meets the intent of the ONR SAPs. For the substantiation of hydrocarbon fires, further work will be required in the detailed design phase. Overall, for these sample areas, I am satisfied that the RP’s demonstration has fulfilled the purposes of GDA.

4.9 Application of Design Principles and Methods – Sample 5 – BEX (SSE2 Structure)

577. The two non-safety classified (NC) structures within the GDA scope (Equipment Access Building BEX and Personnel Access Building BPX) are smaller in size and have simpler configurations than other buildings, and a seismic categorisation of SSE-2. The purpose of sampling the BEX is to: confirm the differences in analysis and design methodologies to those used for the Class 1 structures; and to seek assurance that the design of these non-classified structures does not adversely affect the safety functional requirements of safety critical structures, systems or components (SSCs). As noted in paras. 236 and 255 above, detailed member design is outside the declared

GDA scope. I consider this a reasonable exclusion for the purpose of GDA that can be dealt with as normal business in the site-specific phase.

578. The information presented in Section A.1 of Annex 5 of this report describes the site arrangement, illustrating the locations of the BEX and BPX buildings. The BEX is a simple reinforced concrete structure with an independent raft foundation immediately to the west of the common raft foundation. The structure is a single room on plan, with external walls, one internal floor and a roof slab. The structure has openings to allow large equipment to be lifted from the 0.00m AoD level to the +17.50m AoD level.

579. From my review of the RP's documentation on the BEX, I note that the BoSC (Ref. 37) provides a high level SFR schedule. The SADR (Ref. 66) provides calculations for the global stability and seismic joint checks. These calculations are intended to demonstrate that the BEX will not impact the neighbouring SSE-1 structure (the BSA building), as outlined in Ref. 12.

4.9.1 Global Stability

580. The seismic analysis has used a very soft (V_s of 150m/s) soil profile, and this is expected to generate the largest seismic displacements to feed into the seismic joint checks. I note that this GDA approach will not envelope the seismic demands on the BEX structural members for the full range of GDA soil conditions[37]. Further, the potential for overturning under seismic loading has a factor of safety above the acceptance criteria, which could result in uplift of the foundation in a medium soil profile. Given the nature of this facility, I am content to view this as a minor shortfall that can be resolved as normal business in the site-specific phase.

581. The methods for assessing global stability are discussed in Section 4.4.7 above. During my assessment, I identified the BEX calculations have a low factor of safety for sliding analysis. I further note that the sliding checks are sensitive to the backfill, yet to be specified, with the assumed soil properties outlined in the SADR. This will be presented at the site-specific phase as part of normal business, as will any temporary loading conditions e.g., increased compaction pressures depending on the backfill process implemented on site. This is related to the methodology outlined in paragraph 235 and assessment finding AF-UKHPR1000-0218. I expect that in addressing the finding, the RP will follow RGP e.g., when undertaking sliding checks using ASCE43-19, the guidance assumes the structure has no contact with the soil in the top half of the embedment. Further, I note the requirement for the dynamic soil pressures to not occur simultaneously on opposite walls in a seismic event, as is currently presented by the RP. The RP has indicated that, if the global stability exceeds the acceptance criteria, a shear key, or other design adjustments, can be used. For GDA, I accept that such means of design adjustment in future will produce a credible design that aligns with RGP. I am content that this matter constitutes normal business.

4.9.2 Seismic Joints

582. During my assessment, I identified that the seismic joint check is required to be updated to accommodate the changes in RGP (revision of ASCE43-05 to ASCE43-19). This change states that elastic displacements for Limit State C structures with a fundamental frequency greater than 1Hz shall be increased to account for the higher ratio of inelastic displacement associated in such structures. The BEX has a natural frequency greater than 1Hz, even including SSI with the very soft (V_s of 150m/s) soil profile. The RP stated that the elastic displacement would be increased by a factor of 1.8 for the seismic joint check of limit state C structures at the site-specific phase, but

[37] I expect larger seismic actions for the stiffer soil profiles within the GDA site envelope, which would increase the action and could reduce the resistance in the stability calculations.

Ref. 24 has not been updated to include this information. I regard this as a minor shortfall that can be resolved as normal business.

583. Ref. 24 states that, if the analysis of the SSE-2 structures determines the lateral displacements exceed the design limits, the building could be re-designated to Limit State D. Member design checks are outside the scope of GDA, so it is not clear what impact this would have on the BEX civil engineering design.

584. Seismic joints have been sampled in more detail for the BNX (see Section. 4.8.3). The calculations provided for the BEX (Ref. 66) consider the displacement due to seismic actions. From my assessment, I note that the calculations for the seismic joints for BEX do not consider: the relative seismic displacement of the BEX foundation to the common raft foundation; the static horizontal displacement due to ground settlement and tilt; or the increase to account for inelastic seismic displacements. Nevertheless, the RP has stated that increasing the width of the seismic joint would not have a significant impact on the design; therefore, I am content this can be resolved as normal business in the site-specific phase.

4.9.3 Strengths

585. During my assessment recorded above, I have noted the following strengths:

- The RP has proposed credible design modifications that can be employed if necessary, to ensure that global stability requirements can be met as the design develops.
- Seismic joint widths are not critical to the layout and can be increased if necessary, during the site-specific design.

4.9.4 Outcomes

586. In summary, from my assessment of the BEX recorded above, I am satisfied with the differences in design methodologies for SSE2 structures not on the common raft foundation. The evidence reviewed provides adequate assurance that this and other similar non-safety classified structures will not threaten the function of adjacent safety critical facilities, as per the intent of SAP ECE.1.

587. I have raised no findings but have identified some minor shortfalls and normal business items in the above sections.

4.9.5 Conclusion

588. Based on my assessment of the BEX above, for GDA I am content with the RP’s demonstration of their design principles and methodologies. I consider that the evidence presented is in accordance with RGP and meets the intent of the ONR SAPs. Therefore, for the BEX sample area, I am satisfied that the RP’s demonstration has fulfilled the purposes of GDA.

4.10 Application of Design Principles and Methods – Sample 6 – Malicious Aircraft Impact Protection

4.10.1 Design Requirements and Parameters

589. The main objective of the RP’s aircraft impact assessment is outlined in Ref. 20. This defines the fundamental safety functions (corresponding to performance requirements in NEI-07-13_8P, and in line with IAEA SF-1) as follows:

- The reactor core remains cooled, or the containment remains intact.
- Spent fuel cooling and spent fuel pool integrity is maintained.

- As required in IAEA SRS86, at least one safe shutdown and heat removal path should be guaranteed for malicious aircraft impact.

Although not explicitly defined in Ref. 20, the RP has considered an additional requirement to limit the effective dose to a person off-site. The acceptance criteria, in terms of an effective dose, is defined in the ONR expectations letter, see Ref 126. These fundamental safety functions have been assessed by the ONR External Hazards Inspector, the outcome of which is reported in Ref. 55.

590. The RP has broken down these fundamental safety functions into more detailed low-level safety functions (LLSF's). These LLSFs (E1-4. E2-4, H4-2, C3-2, C3-3) are presented in Table T-4-1 of Ref. 129. From my assessment I am content with the decomposition and definition of these LLSF's.

591. As described in Section 4.2 above, based on these LLSFs, the RP has developed a comprehensive schedule of SFR's applicable to each structure; these are presented in the BoSC documentation for individual structures. I note that the safety function requirement (SFR) schedules do not, however, cover beyond design basis conditions; therefore, the aircraft impact SFRs are omitted. They are instead presented in Appendix A of Ref. 129 for all affected structures.

592. From my assessment of the aircraft impact SFR's within the SFR schedule, for GDA I am satisfied that this structuring of the AIA SFRs is reasonable, but highlight the comment made regarding improving these schedules for the site-specific phase in paragraphs 151 and 274 above. I note the RP has included an "Upstream Reference" column in the schedules, to aid traceability back to Ref. 20, which identifies the structures upon which specific requirements are placed. The RP has substantiated the requirements in the SFR schedule in Ref. 88. From my review I am satisfied for the purpose of GDA that the SFR schedule provides sufficient visibility and detail of the SFRs to capture and trace the requirements being placed on specific parts of the structure. Further, I am content that the DSR adequately demonstrates how the SFRs have been met.

593. With respect to the codes and standards applied by the RP, these are outlined in Section 3 of Ref. 20 and are summarised as follows:

- IAEA NS-G-1.5 provides guidance on the development of the accidental aircraft crash Load Time Function.
- NEI 07-13_8P provides acceptance criteria for malicious aircraft crash and the evaluation methodology for global, local, vibration and fire effects.
- IAEA SRS86 provides acceptance criteria and protection measures for malicious aircraft crash.
- IAEA SRS87 provides the evaluation methodology for global, local, vibration and fire effects.
- IAEA SRS88 provides the margin evaluation methodology and strategies for enhancing the margin – considered in the cliff-edge evaluation.

I note that there is some overlap between the use of the IAEA SRS documents and NEI 07-13_8P, and the documents do not always align. In such instances[38], the RP has accounted for the requirements of both NEI 07-13_8P and IAEA SRS87. However, generally there is no conflict between the two, and often the IAEA SRS documents refer to NEI 07-13_8P or they expand on it and provide more detail. In summary, I am content that the RP is selecting RGP appropriately.

[38] *An example of an area of overlap, where the IAEA SRS guidance and NEI 07-13_8P do not align is for the methodology for many aspects of the damage evaluation (e.g., structural consequences, vibration, fire spread),*

594. The RP's methodology and how the ONR assessment is divided between External hazards and Civil Engineering is shown in Table 8 below.

Table 8: RP's methodology for malicious aircraft impact. This table describes the division of assessment between ONR Civil Engineering and External Hazards

Methodology	ONR Discipline
Definition of the malicious aircraft threats in accordance with ONR's expectations. These are defined as an aircraft model with associated impact velocity, mass and impact angle as per Ref. 137.	External Hazards (Section 4.12.1.3 of Ref. 55)
Identify the safety systems required to achieve the Fundamental Safety Functions for the aircraft impact assessment along with their supporting systems.	
Determine the extent of the aircraft impact protection based on the location of the identified safety systems.	
Examine the site layout and shielding provided by surrounding buildings to determine possible impact locations for analysis.	
For each of these impact locations develop bounding impact scenarios and evaluate the damage to SSCs via analysis and characterisation of local structural damage and consequential hazards, global structural effects, vibration levels and fire damage.	Civil Engineering
Evaluate the ramifications for SSCs based on the predicted damage arising from each impact location to determine whether the acceptance criteria can be met and where necessary modify the design accordingly.	External Hazards (Section 4.12.1.3 of Ref. 55)

595. Section 5.4 of Ref. 20 provides further details on the civil engineering methodology, summarised below:

- Global structural effects are analysed by applying malicious aircraft impact load time functions to finite element models in LS-DYNA. Material strains and deflection limits are assessed to determine whether walls and roofs can withstand the malicious aircraft impact load.
- Vibration effects are analysed using finite element models in LS-DYNA. Acceleration time histories are extracted from the finite element models and these are used to evaluate whether equipment would be damaged from the impact event.
- Local structural effects are analysed by comparing wall thicknesses to the required thicknesses from empirical formulae.
- Fire effects are analysed by identifying the location of fire barriers to determine the extent of the area damaged by fire.

596. The RP has analysed these effects for several impact points on the buildings that are covered by the aircraft impact protection shell. The RP has stated in Section 5.2 of Ref. 20 their main assumptions as follows:

- The aircraft crash occurs when the unit is in normal operation states (e.g., power operation, shutdown),
- Aircraft crash and other external hazards within GDA scope are not considered to occur at the same time,
- For buildings not designed against malicious aircraft crash loads, all systems and components inside the buildings which are within the impact zone are assumed to be damaged if this building is impacted by malicious aircraft.

These assumptions are assessed in ONR's external hazards assessment at Ref. 55 which concludes the assumptions are in line with RGP.

597. I note that the RP's assessment has been carried out on a best-estimate basis that has removed many conservatisms to demonstrate that the analysis is realistic.

Nevertheless, the RP has complied with Section 1 of NEI 07-13_8P and adopted the conservatisms listed to offset acknowledged uncertainties. For the purpose of GDA, I am content this is in line with ONRs expectations for assessing this hazard.

598. The material properties adopted by the RP are documented in Ref. 138. These are based on the concrete and steel reinforcement grades consistent with those adopted for the RP's main civil engineering analysis. In line with ONR's expectations and the guidance of NEI07-13_8P, the RP has adopted best-estimate material properties for the aircraft impact analysis. The concrete compressive strength has been taken as the mean compressive strength of Class C40/50 concrete according to BS EN1992-1-1. The steel reinforcement yield strength has been taken as the mean yield strength reported by the UK Authority for Reinforcing Steels (CARES) for B500C. I am content that these choices of best-estimate properties are appropriate.

4.10.2 Assessment of Structural Damage: Local Element Design and Analysis

599. For the local assessment of structural performance, the RP has assessed scabbing, perforation and punching shear. Penetration[39] and spalling[40] are not assessed by the RP, as these do not allow damage to penetrate beyond the impacted wall. I consider this reasonable and am content that the RP is covering the aspects expected in line with Section 2.1.1 of NEI 07-13_8P and Section 5.3.3 of IAEA SRS87. The RP's methodology for calculating the minimum wall thickness required to prevent scabbing, perforation and punching shear is outlined and assessed in the following paragraphs. For context, I note at the outset that all external walls of the aircraft impact protection shell have a minimum thickness stated in Figure F-8.1-3 of Ref. 138.

600. The RP's approach for scabbing is outlined as a list of four formulae in Table T-6.1-2 of Ref. 138. The required thickness to prevent scabbing is stated in Section 9.1.1 of Ref. 138. For my assessment I have not requested any additional information relating to the scabbing formulae, as I am content that the required thickness to prevent scabbing is typically smaller than the required thickness to prevent punching shear (see Section 9 of Ref. 138), which governs the wall thickness required to prevent malicious aircraft impact. Furthermore, I note that all exterior walls of the BSC, BRX and BFX are thicker than the scabbing thickness.

601. The perforation thickness is used for defining the walls which provide shielding[41] from impact. Shielding can be provided by a single wall which is thicker than the perforation thickness. Alternatively, if there are multiple walls which are thinner than the perforation thickness, then a calculation can be made of the residual velocity after the missile has passed through each wall, demonstrating where the missile finally comes to rest. Therefore, the calculation of the perforation thickness is a particularly important part of the Aircraft Impact Assessment. From my assessment of the RP's approach to perforation, I note the following:

- Five potential formulae are listed in Table T-6-1 of Ref. 27. Following an appraisal of these, the RP has applied the 'CEA-EDF' formula[42]. This is the only formula used by the RP to calculate the perforation thickness.
- As recommended in Section 5.3.3.3 of IAEA SRS87, the RP has applied a reduction factor of 0.6 due to the empirical formulae having been derived from solid missiles. This factor is included to account for the deformability of the aircraft engines. I am content with this approach.
- I note that Section 5.3.3.3 of IAEA SRS87 suggests that a safety coefficient of 1.2 should be applied to the results of the formulae to cover test uncertainty.

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[39] Penetration refers to the depth of the crater formed at the zone of impact.
[40] Spalling refers to the ejection of target material from the front face of the target whilst scabbing refers to the ejection of concrete from the distal face.
[41] Shielding in the AIA section 4.9 refers to the shielding of structures to prevent malicious aircraft impact as opposed to shielding that provides radiation protection.
[42] The acronym 'CEA-EDF' is the RP's reference name for this particular formula

However, the RP has confirmed that this factor would not be applied, as aircraft impact is considered a beyond design basis event.

- The RP clarified in response to RQ-UKHPR1000-1004 (Ref. 6) the result of the perforation calculation, which is less than the perforation thickness formally stated in Section 9.1.1 of Ref. 138. I note that this perforation thickness stated in Section 9.1.1 of Ref. 138 was derived earlier in the GDA process, when the calculation method had not been fully developed; nonetheless the RP has decided to retain this result. I am content that this is a conservative approach.

In summary, from my review, I consider that the factor of 1.2 specified in Section 5.3.3.3 of IAEA SRS87 should be included in the RP's methodology for the perforation calculation. I consider this a minor shortfall that can be resolved as normal business in the detailed site-specific design. Nonetheless, the RP's choice of perforation thickness provides an adequate safety factor and thus can be considered bounding. Therefore, for the purpose of GDA, I am satisfied with the perforation calculations.

602. The RP states the minimum thickness to prevent punching shear damage in Section 9 of Ref. 138. Ref. 27 & 138 states that this check is carried out using a Two Degree of Freedom method, in line with the guidance in Section 4.2.2 of IAEA SRS87. The RP presented details of this method of calculation in Workshop #03 (see Table 1 above and Ref. 10). I note that the RP has validated the wall thickness estimated using the Two Degree of Freedom method through finite element analysis, as part of the global structural damage assessment. From my review, whilst I am content with the RP's approach and the results provided in workshop #03, the RP has not reported this information, as the inputs to the calculations include sensitive nuclear information (SNI). Whilst I understand the challenges of dealing with SNI, I consider that the methodology, results and validation should be fully reported using an appropriate means. I am content this is a minor shortfall that can be resolved as normal business in the detailed site-specific design.

4.10.3 Assessment of Structural Damage: Global Structural Design and Analysis

603. The RP has analysed the global structural damage by applying malicious aircraft impact load time functions to finite element models of the structures. The material strains and deflections are then compared to limits, to determine whether these structures can withstand the loads.

Acceptance Criteria:

604. The structural acceptance criteria in NEI 07-13_8P are defined in terms of strain limits for steel (including reinforcement). NEI 07-13_8P does not provide the failure criteria for concrete, and instead references a safeguarded document where the failure criteria are provided.

605. The RP has chosen different acceptance criteria (e.g., support rotation limits) for different aircraft threats. This is shown in Table T-5.2-1 of Ref. 138, where the RP, using guidance from IAEA SRS87, has decided to assess the military aircraft and large commercial aircraft as Design Extension External Event Level 1, and the large cargo aircraft as Design Extension External Event Level 2. In IAEA SRS87, events are assigned as either Design Basis External Events (DBEE), or Design Extension External Event (DEEE) levels 1 or 2, with DEEE Level 2 reserved for the most extreme events. Plant acceptance criteria and structural acceptance criteria are varied depending on whether an event is DBEE, DEEE Level 1 or DEEE Level 2. DEEE Level 2 events only require one means of reactor shutdown or core cooling, whereas DEEE Level 1 events require two means of each. Since the RP's performance criteria from NEI 07-13_8P only require one of each, it is reasonable to conclude that DEEE Level 2 is the most appropriate choice for malicious aircraft impact. Furthermore, I note that IAEA SRS86 recommends applying a tiered approach, with less onerous acceptance

criteria for the most extreme threats. However, I am not convinced that the large cargo aircraft represents a more extreme threat when compared to the two other aircraft types. From my assessment, I am satisfied with the acceptance criteria used by the RP but consider that the acceptance criteria for the large cargo aircraft could have been applied to the other aircraft types. Nevertheless, for the purpose of GDA I am content that the RP has applied greater conservativism here than I would consider necessary.

Choice of Impact Locations:

606. Many different impact locations are possible; however, the damage effects will be bounded by a select number of cases. I note that Section 4.1 of IAEA SRS87 suggests that a limited number of impact cases should be considered in the global structural damage assessment. The impact locations should be chosen to maximise damage. Furthermore, Section 2.4.1 of NEI 07-13_8P states that impacts on the containment should be assumed to occur perpendicular to the centreline, imparting the largest force on the structure. Section 2.4.2 of NEI 07-13_8P suggests the assessment should consider impacts on the building housing the spent fuel pool at the mid-height and mid-span of the wall, with impacts assumed to be normal to the wall. The RP has selected a limited number of impact locations and have not analysed all aircraft types for each impact location. The selected impact cases are as follows.

- BSC: Impact locations include one on the roof, one in the middle of the wall facing the BMX and one on the HVAC inverse-L protective structure. The RP has not considered the cargo aircraft for the impact on the roof because they state that the results would be bounded by the large commercial aircraft as the load time function of the large commercial aircraft has a higher peak force. I am content with the RP's claims and arguments.
- BRX: Impacts are taken on the cylindrical section of the external containment, on the water tank for the Secondary Passive Heat Removal System and the hemispherical dome. The RP has not considered the cargo aircraft for the impact on the dome. The RP states that the results would be bounded by the large commercial aircraft, as the load time function of the large commercial aircraft has a higher peak force. I am content with the RP's claims and arguments.
- BFX: Impact locations are chosen on three of the largest spanning walls and one impact on the roof. The RP has chosen the military aircraft impact for the roof, as the RP claimed the impacts on the roof of the BSC showed that it produced higher deformations. Impacts on the East and West walls considered the large commercial aircraft, as this produced the largest deformations for the impact on the South facing wall. I am content with the RP's claims and arguments.

607. In summary, the RP's justification for not modelling all aircraft types for all impact locations has been provided in Section 7.2 of Ref. 138. From my review, I am satisfied with the arguments made. I consider that the RP's choice of impact locations for the global structural damage assessment represent those which would produce the most structural damage and therefore bound all other impact locations.

Modelling

608. The material model used in LS-DYNA has been subject of expert review recorded at Ref. 29. Considering the outputs of this review, I am satisfied that the RP's concrete material model and input parameters are appropriate and consistent with Section 2.3.4 of NEI07-13_8P. Furthermore, the RP has used sensitivity studies to justify assumptions made, such as the 'ERODE' parameter in LS-DYNA, which I consider appropriate in line with SAP ECE.14.

609. With respect to the mesh size of the models, I note that the solid element size for the walls subject to the impact area is reported in Section 8.1.2 of Ref. 138. Table T-8.1-1 of Ref. 138 confirms the mesh resolution near the impact zones. I note in Section AA.2 of Ref. 138 that the RP has demonstrated that the failure energy is not dependent on mesh size through simple single-element tests. Furthermore, the RP has demonstrated that the maximum mesh size is greater than the limits specified in IAEA SRS87 for solid elements to prevent filtering of vibrations. Although there are no mesh sensitivity studies reported in the RP's documentation, based on the expert advice recorded in Ref. 29, for GDA I am satisfied that the mesh densities described in Ref. 138 are adequate. Furthermore, the sensitivity study carried out on the element formulation provides confidence that the results would not be sensitive to mesh density.

610. The impact analyses were carried out with constant stress elements[43]. This element formulation is less computationally demanding compared to others, but it can result in unrealistic deformations known as hourglassing[44]. The RP has run an additional analysis case with fully integrated solid elements[45] to test the sensitivity to this. Whilst I note that some differences are present in the results in Section AA.6 of Ref. 138, the peak support rotation is similar between the two cases. I note the minimum principal strains in the concrete are shown as slightly lower for the constant stress elements. Overall, I am satisfied that the results are relatively insensitive to changes in element formulation and I consider the element formulation to be adequate for the purpose of GDA.

611. The RP uses separate models for impacts on the different buildings, as shown in Table T-8.1-1 of Ref. 138. I consider this appropriate for the purpose of GDA, as it means that increased detail and mesh resolution can be added near the impact zone, with more coarse mesh and boundary conditions further away.

612. For the BFX impacts, the models have a simplified representation of the safeguard buildings and the BRX. These adjacent buildings are modelled as single masses on beam elements. For the BSC and BRX impacts, the models have a coarse shell element model of the BFX. The models have been refined in different areas, depending on the impact location. A separate model was used for the impact on the roof of the BFX. This is because the roof structure is made up of a concrete slab on top of steel beams, with shear studs between the two. These shear studs have been explicitly modelled, which greatly increased the number of elements, and therefore it was necessary to limit the size of the model to just the roof. I consider that the RP's model simplifications are appropriate for the purpose of GDA and allow sufficient resolution to be applied to the impact area.

613. I note that all models have springs and dashpots added to base nodes, to represent the stiffness and damping of the soil. I judge that the results of the analysis will not be highly sensitive to the soil modelling, as the aircraft impacts are relatively high up the building elevation, and most of the deformation is seen local to the impact. Therefore, for the purpose of GDA I am satisfied with the RP's approach to modelling the soil.

614. With respect to validation, the RP has carried out validation against the Meppen II-4 test[46] where the analysis of deformable missiles impacted into a 700mm thick reinforced concrete wall. I consider that the results from this test case show reasonable agreement in terms of displacements. I am content that this test case demonstrates that the finite element modelling methods used by the RP are valid for representing bending failure under low velocity missile impact. For the purposes of GDA, I am

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[43] *These are denoted 'ELFORM=1' in LS-DYNA.*
[44] *Hourglassing is a state of strain or deformation that is free of energy, known as a zero-energy mode, that can produce spurious results.*
[45] *These are denoted 'ELFORM=-1' in LS-DYNA.*
[46] *Experimental and theoretical investigations on the impact of deformable missiles onto reinforced concrete slabs*

satisfied that the RP has carried out appropriate validation in accordance with both IAEA SRS87 and NEI07-13 8P.

Results of Global Structural Damage Assessment

615. The results from the RP's global structural damage assessment indicate that most of the analysis cases pass the acceptance criteria on support rotation limits. I note that many cases which are assessed to Design Extension External Event Level 1 acceptance criteria (see paragraph 605 above) have regions that are above the concrete strain limit. For most cases, there are only a small number of elements above the strain limit, and therefore I am satisfied with the results. Cases MA500 and CA3000 are the exception to this and are discussed in more detail below:

616. Case MA500 assesses an impact on the inverse-L protective structure on the exterior wall of the BSC. The maximum support rotation for this case is 4.47°. This structure is a vertical wall that does not carry any vertical compressive load and therefore it is appropriate to use the support rotation acceptance criteria for slabs. The RP has clarified in response to RQ-UKHPR1000-1333 (Ref. 6) that the Design Extension External Event Level 2 criteria will be used for this case (this has no concrete strain limit and 6° maximum support rotation for slabs). Table T-5.2-1 of Ref. 138 shows that the acceptance criteria of Design Extension External Event Level 1 applies to all military aircraft cases. I note that Section 9.2.15 of Ref. 138 states that case MA500 could meet the acceptance criteria for Design Extension External Event Level 2, but it does not explicitly state that this acceptance criteria are being applied to this case. Therefore, it does not appear that this case meets the acceptance criteria which have been set by the RP.

617. Case CA3000 assesses an impact on the dome of the external containment by the commercial aircraft. The results for this case show that there is a large region at the joint between the dome and the cylindrical wall where the minimum (most compressive) concrete principal strain is above the acceptance criteria. This region extends through the thickness of the concrete and is claimed to be due to arching action. The RP clarified in response to RQ-UKHPR1000-0904 and RQ-UKHPR1000-1333 (Ref. 6) that the acceptance criteria of Design Extension External Event Level 2 would be applied to case CA3000.Table 5-2.1-1 of Ref. 138 shows that the acceptance criteria of Design Extension External Event Level 1 applies to all large commercial aircraft cases. Section 9.2.22 of Ref. 138 states that case CA3000 could meet the acceptance criteria for Design Extension External Event Level 2, but it does not explicitly state that this acceptance criteria is being applied to this case. Therefore, it does not appear that this case meets the acceptance criteria which have been set by the RP.

618. For both cases CA3000 and MA500, I am content that the results would meet the Design Extension External Event Level 2 acceptance criteria. As noted in paragraph 605 above, I consider this less onerous acceptance criteria to be appropriate. Therefore, I am content that this inconsistency can be resolved as normal business in the site-specific phase and is not significant for the purposes of GDA.

4.10.4 Assessment of Structural Damage: Global stability

619. With respect to global stability checks, the RP has carried out checks for the BFX on the basis that this building is relatively tall and narrow when compared to the other buildings; therefore the results should bound those for the other buildings. I am content with this rationale for the purposes of GDA. From my review of the methodology, I note that the RP assumes the BFX is founded on a separate raft with no embedment considered. I consider these to be conservative assumptions, in line with SAP ECE.13. From my assessment of the results, I am content that the acceptance criteria for the overturning and sliding resistance checks are met. I note that the RP has included a

check on the bearing pressure, although this is not deemed necessary in IAEA SRS 87. Overall, I am satisfied that the GDA demonstration for global stability is adequate. I expect the RP to implement this methodology for other applicable facilities in the site-specific phase and consider this to be normal business.

4.10.5 Other Structural Damage Analysis

620. The RP has designed a structure which is located over the seismic joints (or gap) between the BSC and the external containment. The purpose of this structure is to protect the joint and prevent fuel and fire from entering through it in the event of an aircraft impact. The analysis of this structure to demonstrate that it can perform as required is described in Appendix X of Ref. 138. The finite element analysis models of the threats that were used for the generation of the load time functions were impacted into the structure. The aircraft fuel was represented as Smoothed Particle Hydrodynamics (SPH) elements. The analysis showed that the initial design for this feature incurred significant damage and SPH elements travelled into the seismic gap. The original design is shown in Figure F-X.1-3 of Ref. 138 which was modified to that which is shown in Figure F-X.5-2 of Ref. 138. Subsequent finite element analysis of this update shows less damage is sustained, with no SPH elements travelling into the seismic gap. From my assessment, I am content that the analysis has been carried out appropriately, resulting in a suggested improvement to the GDA design. However, I expect such an improvement to be implemented in the site-specific phase and any other seismic joints to be appropriately considered. This requirement for further work is captured in assessment finding AF-UKHPR1000-0235 below.

AF-UKHPR1000-0235 – The licensee shall, as part of the detailed design for malicious aircraft impact, demonstrate that the protective structures over the seismic gaps prevent fuel from penetrating between the buildings. This should include, but not be limited to, the gap between Safeguard Building C and the External Containment.

621. As noted in paragraph 617 above, the finite element analysis results for the impact onto the dome of the containment (case reference CA3000) showed a large area with relatively large concrete strains. This potentially poses a consequential hazard from concrete spalling falling onto SSCs located in the gap between the internal and external containments. The RP clarified in Section 11.5 of Ref. 138 that the different trains of the systems cross the gap between the internal and external containment at different locations around the circumference. Therefore, the RP claims that falling concrete from one impact location could not damage all three trains of any system. Furthermore, the RP indicated that there will be steel walkways between the internal and external containments which would mitigate spalled concrete falling onto safety critical SSCs below. From my assessment, I consider the arguments presented by the RP to be adequate, and that the design of these walkways will need to consider this further in the site-specific phase. Moreover, I highlight that these steel walkways could present a path for vibration that the RP has not considered in the vibration damage assessment. I consider these two points to represent minor shortfalls that can be resolved as normal business in detailed design.

4.10.6 Vibration Damage

622. The RP's methodology for the vibration damage assessment is articulated in Refs. 139 and 14. Vibration damage has been assessed by recording accelerations in finite element models and comparing these to vibration limits for different types of equipment. Acceleration time histories have been extracted from selected nodes on each floor level. The locations of these nodes are shown in Figures F-4.1-1 to F-4.4-9 of Ref. 139. They have been placed close to equipment locations, or at the mid span of floors where the vibrations would be expected to peak. The time histories of the

acceleration have then been used to create response spectra for each floor level. The response spectra from each floor have then been enveloped across all output locations on that floor. The peak pseudo-acceleration from the response spectrum is then compared to the median fragility limits provided in Table 3-3 of NEI 07-13_8P. If the peak from the response spectrum is lower than the median fragility limit, then the RP assumes that the equipment is undamaged.

623. From my assessment of this methodology, I note that there are differences between the method chosen by the RP and RGP as described in NEI 07-13_8P. The RP has not filtered the time histories at 200Hz for comparison with the limits in Table 3-3 of NEI07-13_8P. Instead, the RP has used the peak from the response spectrum for the comparison. I am content that this will add conservatism into the RP's results and, therefore, I consider the RP's methodology to be adequate for the purposes of GDA.

Choice of Impact Locations

624. The impact locations have been chosen by the RP to maximise vibrations experienced by the equipment. Impact locations on walls are typically placed in line with the slab levels, to minimise the distance of the path through the structure from the impact location to the equipment locations. I am content with this approach.

625. From my assessment, I note that the impact locations on the fuel building (BFX) are the same as those used for the global structural damage assessment. These locations are at the mid-span of the walls, as they have been selected to maximise the out of plane bending of these walls. The RP has justified this in Appendix D.2 of Ref. 139, claiming the following:

- The equipment located higher up in the building are pipes and pumps which are likely to have higher acceleration limits when compared to other types of equipment.
- The walls of the lower floors of the building are shielded by nearby buildings, such as the BEJ, BNX, BRX and BSA/BSB.

626. I noted that in Ref. 14, the vibration effects on some other BFX systems were mentioned (i.e., REA, SED, RBS, ASP). The RP has shown in Ref. 139 that these systems are located below +0.0mAoD and has confirmed that that impact locations lower than those considered are unlikely to affect these systems. I am content with the RP's arguments for these systems. In summary, I am satisfied with these arguments and am content with the RP's choice of impact locations used for the BFX and the other facilities.

627. To reduce the number of cases analysed, the RP has not simulated impacts of all aircraft types at all locations. Rather, the large commercial aircraft has been used for most impact locations. The RP's justification for this is provided in Appendix D.1 of Ref. 139 and is based on analysis at location LC23[47] on the +13.2m level of BSC. The RP argues that the location LC23 is the critical case for vibration qualification and therefore all three aircraft types have been modelled at this location. The RP's results show that the peak accelerations from the large commercial aircraft were larger than or equal to those from the military aircraft. From my assessment, I am satisfied with these simplifications the RP has made.

628. For the impacts on BSC, I note that the RP has considered +13.2mAoD to be the lowest that an aircraft can impact the building, due to the shielding claimed to be provided by the BMX building. From my assessment, I note that no appropriate shielding arguments have been provided for the shielding of BSC by BMX. Therefore, either lower impact locations need to be considered in the analysis, or the shielding

[47] *LC## is a referencing system used by the RP to denote specific locations within the buildings*

claims substantiated. I consider that this could be significant to the performance of the MCR and RSS system, noting that the RSS is located at +8.7m. This will need to be resolved in the detailed site-specific design and is recorded in assessment finding AF-UKHPR1000-0092 in the ONR external hazards assessment report (Ref. 55).

Results of Vibration Damage Assessment

629. From my assessment of the results presented in Section 7 of Ref. 139, I note that the only areas where the peak of the response spectra is above the relevant limit provided in Table 3-3 of NEI 07-13_8P are:

- BSC Level +8.7m
- BSC Level +13.2m
- BSC Level +17.6m
- BSC Level +21.8m

630. All other floors of the BSC have accelerations below the relevant limit. The RSS is located on Level +8.7m; however separate response spectra have been generated for this room, and this room is below the limit. The RP has therefore assumed that the vibration damage from an impact on the BSC results in loss of:

- MCR
- Train C of ASG [EFWS]
- Train C of RIS [SIS]
- Train C of RBS [EBS]

631. From my review of the systems used for safe shutdown or heat removal identified in Table T-5-2 of Ref. 20, I note that these are the only frontline safety systems which are located within BSC. Other trains of these systems are available in the other safeguard buildings, and the plant can be shut down from the RSS. Therefore, I am content that the results from the vibration damage analysis are adequate.

4.10.7 Fire Damage

632. The assessment of the fire spread into and through the buildings using appropriate fire spread rules is covered in Ref. 55. In this section, I assess the civil engineering related aspects of the fire barriers.

633. With respect to the qualification of the fire rated barriers, the RP has presented analysis to show that the concrete walls on these boundaries can withstand a 3-hour fire. I am content with this, noting that the detailed analysis will need to be carried out in the site-specific phase. Furthermore, the RP has now specified explicitly in the SFR schedule that fire barrier members will be rated for a 3-hour fire, see Ref. 129 and paragraph 592 above.

634. For other fire barriers that remain 2-hour rated, such as those in BFX, the RP has summed the fire barrier ratings along a given fire spread path as described in Section 2.4 of Ref. 14:

"When multiple fire barriers are applied on the fire spreading route, the first fire barrier is assumed to be damaged by the overpressure induced by aircraft crash. If the total fire-resistant capacity of the unaffected fire barriers is greater than 3 hours (the space between the first two fire barriers is greater than $56.6m^3$), the fire is assumed to be stopped from further propagation;"

Based on expert advice recorded at Ref. 29, I am content that this is an appropriate way to apply the 3-hour fire barrier rules from NEI 07-13_8P to a design that has 2-hour rated barriers.

635. With respect to fuel ingress through cracks, in Ref. 138 the RP has demonstrated using the results for the strain in the shear reinforcement from the finite element analysis modelling that through thickness cracks would be very small (~2mm). Considering the thickness of the external walls of the aircraft impact protection shell, the RP concludes that this does not present a significant path for fuel and fire spread. I am content with this argument.

4.10.8 Strengths

636. During my assessment recorded above, I have noted the following strengths:

- The RP has identified and applied internationally recognised RGP for the design of the structures providing aircraft impact protection.
- The RP has defined and documented a clear methodology for the design and analysis of the aircraft impact structures. This methodology is appropriate and, where simplifications or analyst judgement has been required, a conservative approach has been taken that aligns with proven engineering practices.
- The RP has systematically defined the design parameters applicable to the design of aircraft impact structures and these are consistent with parameters used for other forms of structural analysis.
- The RP has sufficiently verified and validated the methods, including analysis and design tools, and design inputs and outputs, using suitably independent methods and studies.

4.10.9 Outcomes

637. In summary, from my assessment of the malicious aircraft impact protection recorded above, I am content that the RP has presented a thorough demonstration of the application of their methodology. As highlighted by the closure for Action 3 of RO-UKHPR1000-0007 (see Ref. 104), I am satisfied that the design is robust, and the design provision is sufficient to satisfy SAPs ECE.1 and ECE.2 (specifically SAPs paragraphs 334 and 337).

638. From my assessment, I have raised an assessment finding to ensure the licensee demonstrates that the protective structures over the seismic gaps prevent fuel from penetrating between the buildings. This is detailed in Annex 4.

639. Further to the above, I have identified several minor shortfalls and normal business items in the above sections.

4.10.10 Conclusion

640. Based on my assessment of the malicious aircraft impact protection above, for GDA I am content with the RP’s design principles and methodology and the demonstration of its application. I consider that the evidence presented is in accordance with RGP and meets the intent of the ONR SAPs. Therefore, for the malicious aircraft impact protection sample area, I am satisfied that the RP has fulfilled the purposes of GDA.

4.11 Further Safety Case Considerations

641. This section considers discrete but important aspects of the safety case, including novelty, radiation protection, conservatisms, construction, EIMT and decommissioning.

4.11.1 Assessment

<u>Novelty</u>

642. During my GDA assessment, I have not noted any areas that would be classed as novel design solutions. The civil engineering design has followed relevant good

practice for nuclear power plants and the RP has adopted solutions that are consistent with other nuclear power plants under construction in the UK.

643. I note that there are differences between the generic UK HPR1000 and the FCG3 design that has been constructed in China as follows:

- Different design code bases (from Chinese to American/European).
- Increases to member thickness e.g., walls and slabs for aircraft impact shell and the common raft foundation.
- Changes to building layouts and geometries e.g., BFX building.
- Credible solutions for controlling settlement on a soft soil profile.

644. However, I am content for GDA that none of the above give rise to risks that require highlighting as novel and requiring special consideration in the detailed and site specific design stages.

Radiation Protection

645. As part of the ‘upstream’ derivation of functions, it is my expectation that other disciplines identify the shielding requirements that form inputs to the civil engineering design. The intent of this is to sufficiently prevent or reduce radiation exposure to personnel. As part of the civil engineering design, the civil engineering responsibility is to specify materials for the civil engineering SSCs to minimise activation so far as is reasonably practicable.

646. The RP defines the function of shielding as low-level safety function ‘C6-1, Shield against radiation’. Function C6-1 corresponds to the detailed safety function ‘F4 – function to confine the radioactive materials, shield radiation and reduce radioactive release’. This F4 function corresponds to engineering requirements within the SFR schedules, typically stating: “Ensure walls and slabs are adequately thick to sufficiently impede the radiation from the walls and roofs. The structural members have enough thickness to meet the requirements of radiation shielding”. Whilst I have not reviewed the corresponding radiation shielding design reports, which states the acceptance criteria, I note that the reports schedule thicknesses and materials for each shield. I sampled the radiation shielding design report for the BFX (Ref. 140) to check the assumptions made for the shielding design are consistent with the civil engineering design. As a result of my assessment, I am satisfied that the upstream inputs to design are met by the civil engineering design.

647. The material specification for civil engineering structures that provide shielding are concrete, steel and stainless steel. Whilst there is little evidence of specific consideration for minimising activation, the material selection is based on established practice for similar nuclear facilities. The decommissioning report (Ref. 141) discusses materials susceptible to activation, predominantly non-civil engineering related materials. Of note for civil engineering, this report identifies the need to coat structural carbon steels, low alloy steels and concrete with decontaminable paint, to be tested prior to use inside the internal containment. Details of this test were not reported. This report also discusses radiation shielding requirements during deconstruction, identifying how permanent, moveable and temporary shielding structures will be used. From my assessment, for GDA I am content with the material selection with respect to minimising activation.

648. From my assessment, I am content that consideration and coordination has been given to ensuring the civil engineering structures fulfil the requirements placed upon them related to radiation protection.

Defence in Depth and Conservatism

649. It is my expectation that the design and safety case include margins of conservatism to allow for uncertainties (SAP ERL.4) and that the safety case should identify the areas of optimism and uncertainty (SAP SC.5).

650. Whilst there is little evidence of explicit and direct consideration of defence in depth within the civil engineering discipline, the civil engineering design satisfies the safety functional requirements and follows RGP as outlined in Section 4.12 below. The following areas are of particular note:

- The structures are generally of a multi-compartment form that has inherent structural redundancy with multiple load paths.
- The design is based on an envelope of external hazards considered for FCG3 and the UK context, which has led to inherent conservatism in the design basis inputs (e.g., wind loading).
- The RP has inherited conservatism from partial factors on loads / load combinations and resistances.
- The design has been based on elastic analyses at the design basis.
- Compliance with ACI349-13 Chapter 21 seismic detailing rules should ensure a ductile response.
- The beyond design basis capability has been thoroughly evaluated to demonstrate adequate margins to failure.
- The ultimate capacity of the internal containment has been thoroughly evaluated and adequate margins demonstrated.

651. As a result of my assessment, I am satisfied that the civil engineering GDA design has demonstrated defence in depth and conservatism and has met the intent of SAP EKP.3 and EHA.6. I am content that the detailed design phase will offer opportunities to refine the design of individual elements to confirm code compliance and final design margins.

Constructability

652. For the purposes of GDA, assumptions regarding the construction and durability within the design are assessed as to whether these are achievable and whether they would meet UK CDM requirements. Design decisions at this stage should have consideration for whole-life nuclear safety and conventional health and safety risk reduction. During GDA, I have assessed whether the proposed construction methodologies underpin, inform and are consistent with their design risk assumptions, in line with the expectations of SAP ECE.25.

653. Whilst construction is mentioned in the building specific reports (e.g., BoSC, BoD and DSR), the main discussion is presented in Refs. 75 and 78. These are general reports that present examples. These two reports use examples of constructability from the construction of the reference design at FCG3, where considerations of health and safety have been demonstrated. Refs. 75 and 78 also demonstrate the RP's understanding of the benefits of collaboration between designers and constructors by reference to the liaison between CGN's design and construction teams. Further, the reports acknowledge and demonstrate a theoretical understanding of the UK CDM regulations and make reference to the phrase "the UK Principal Contractor may adopt a different methodology".

654. For the internal containment (see Section 4.6.13) and common raft foundation (see paragraph 509 above and Section 4.7.7), I identified no significant concerns regarding constructability. The BFX roof was sampled to assess its constructability, and this is presented in Section 4.5.9 above. The considerations from the BRX gusset are discussed in paragraph 425 and Section 4.6.13 above, where I note that specific

consideration for constructability is required for design changes, such as including for additional reinforcement, where alternative solutions may be sought if constructability is found to be compromised during the detailed design phase. The considerations relating to the constructability associated with the use of silica fume in the concrete mix is discussed in paragraph 191 above.

655. Ref. 78 includes a design risk register, using the principles of prevention, with risk elimination favoured over risk reduction and use of controls, which I am satisfied with. There is a series of cross-cutting risk registers contained within a suite of building-specific reports held within a risk register which have been sampled and assessed within the conventional health and safety assessment report (see Ref. 131).

656. Based on the elements sampled within my assessment, I can conclude that the RP has adequately demonstrated constructability for the civil engineering structures at a level appropriate for GDA. I note that there is considerable work required to fulfil the UK context and meet the expectations of CDM2015 and SAP ECE.25 at the detailed design and site-specific phases; however, I am content this is normal business.

EIMT

657. During my assessment, I have identified some aspects of EIMT to assess the adequacy of whether these meet the expectations of SAPs ECE.20, ECE.21, ECE.22 and ECE.24.

658. The RP has presented a report that relates to EIMT for civil engineering (Ref. 72). The report defines the EIMT strategy and includes a high-level overview of likely procedures and technologies, including legislation, codes and standards, building instrumentation, containment integrity and leak tightness testing, aging effects (time-based testing) and routine maintenance. The report references examples of the methods developed for and deployed at FCG3.

659. As part of my assessment, I have noted EIMT considerations for the parts of the design that have been presented as part of my assessment. This includes BFX, discussed in paragraphs 318 and 336 above, and the internal containment, discussed in Section 4.6.13 above.

660. For the purposes of GDA, the level of information presented is adequate, as it identifies the methods by which the civil engineering structures will be monitored and maintained. Conventional health and safety of these methods are not presented, and I would expect this to be considered, refined, and presented at the site-specific stage.

Decommissioning

661. As part of my assessment, I expect that decommissioning be considered at the design stage to incorporate design features to facilitate future decommissioning, as appropriate. The adequacy of this section is to judge whether the RP's submissions meet the expectations in SAP ECE.26. For further considerations of the DC-series of SAPs, see the ONR decommissioning assessment report (Ref. 142) and the ONR conventional safety assessment report (Ref. 131) for further considerations of health and safety considerations (including CDM2015) related to decommissioning activities.

662. Each of the BoSC, BoD and DSR reports that I sampled acknowledge decommissioning, with the BoSC reports containing the high-level statement "the UK HPR1000 plant is designed to facilitate safe decommissioning using current available technologies". The RP presents a list of technologies that could be employed in Ref. 79, which I confirm are credible for the intended purpose. Ref. 79 is also identified in the golden thread diagram (see Figure 1, above and in Annex 6) as a key document to demonstrate the golden thread. Ref. 79 contains discussion on the decontamination of

civil structural materials (focussing on concrete), dismantling and breaking up technologies (aiming to minimise volumes at the higher contamination levels). The report does not specify which locations the technologies should be used for, which is an appropriate level of detail for the purposes of GDA.

663. As part of my assessment, I sampled the internal containment structure to assess whether the RP had considered decommissioning during the GDA phase. The civil engineering structures in GDA scope are predominantly standard construction, with few scenarios that would provide challenges at the decommissioning phase. Ref. 79 states a methodology for the demolition of the internal containment, citing relevant operational experience, using existing technologies. In Ref. 79, when discussing building demolition, the RP makes reference to Ref. 143, in the context of the radiological categorisation of waste. Ref. 79 outlines the methodology for building demolition and proposes methods to manage potentially localised irradiation and activation of civil engineering elements. From my assessment, I noted that the RP has presented consideration of access arrangements and aligned these with design measures (including structural geometry and layout, removable plates, barrier shields and access openings) to facilitate decommissioning activities. Further, the RP has presented consideration of bearing capacity of floors for dismantling large components, with the inclusion of temporary shielding and removable floor plates into the design. I note that the internal containment is of similar form and characteristics to other containment designs in the UK. In summary, I am satisfied that the RP's proposals for decommissioning the internal containment outlined in Ref. 79 are adequate for GDA and meet the intent of SAP ECE.26.

664. I note that Ref. 79 references out to other documents, notably Ref. 141, which describes the design features and decisions, such as material selection and layout planning, that have been influenced by operational experience and decommissioning considerations. This report also identifies limitations of the work undertaken to date, e.g., identifying that there may be a future need to evaluation additional loads from any temporary shielding walls used in decommissioning. Furthermore, Ref. 144 (not directly referenced in Ref. 79) collates RGP within the topic area of decommissioning, with a number of requirements that are specific to civil engineering. As an aside, I note that references 145 and 146 are cross-cutting reports, the latter of which supports PCSR Chapter 24 (Ref. 13). Whilst these reports provide some context to decommissioning, these reports provide insufficient supplementary information for the civil engineering aspects to be included in my assessment.

665. From my assessment, I judge that for GDA, the RP has adequately considered decommissioning of the civil engineering structures. The RP's evidence provides assurance that, at this early design stage, considerations for decommissioning are being implemented within the design. Although there is a lack of clarity around the hierarchy of decommissioning documentation, I consider that the information provided is sufficient for the purposes of GDA and meets the intent of SAP ECE.26.

4.11.2 Strengths

666. During my assessment recorded above, I have noted the following strengths:

- The civil engineering design is consistent with RGP for nuclear power plants and avoids novel design solutions.
- The civil engineering design has incorporated features that provide robust defence in depth capability.
- The civil engineering design includes features and considerations for the future decommissioning activities.

4.11.3 Outcomes

667. In summary, from my assessment of the above safety case considerations, I am satisfied that the RP has presented an adequate demonstration for GDA for novelty, radiation protection, conservatisms, construction, EIMT and decommissioning and has demonstrated awareness of what will be need in the detailed design and site-specific phases. Furthermore, I am satisfied the civil engineering design is inherently robust and has demonstrated adequate consideration of defence in depth to meet the intent of SAP EKP.3.

4.11.4 Conclusion

668. Based on my assessment of the further safety case considerations covering novelty, radiation protection, conservatisms, construction, EIMT and decommissioning. I consider that the evidence presented is in accordance with RGP and meets the intent of the ONR SAPs. Therefore, for this sample area, I am satisfied that the RP has fulfilled the purposes of GDA.

4.12 Demonstration that Relevant Risks Have Been Reduced to ALARP

669. For civil engineering structures the demonstration that risks have been reduced ALARP is predicated on applying RGP for the analysis and design process under design basis conditions and consideration of cliff edge and beyond design basis response. This is expected to inform a thorough post-design review to consider whether adding further targeted structural enhancements is grossly disproportionate to the risk reduction potentially achieved.

670. With respect to the application of RGP, from my assessment I have noted areas where improvement is needed and / or future work is required to optimise or more fully articulate site-specific aspects of the methodology. Nonetheless, for the purposes of GDA, I am satisfied the RP has adequately demonstrated the application of RGP and I highlight the following:

- Design inputs to civil engineering that originate from other disciplines have been interpreted and applied appropriately. These inputs have been deemed adequate for the purpose of GDA, see Refs. 55, 41 and 56.
- The declared suite of standards and design codes are both compatible and relevant for their application and have been applied appropriately.
- In areas where the codes are non-specific, ambiguous or lacking detail, the RP has used appropriate and reasoned engineering knowledge and judgement drawing from other sources of RGP, whilst generally ensuring a conservative approach.
- For the GDA demonstration the RP has thoroughly investigated variability, uncertainty and sensitivity, and this has been accounted for in a manner that meets the intent of SAPs ECE.1, ECE.13 and AV.6.
- The design software used in conjunction with the modelling and analysis methodologies have widely accepted technical provenance and reflect RGP. In house software has been thoroughly verified and validated and improvements made.
- Validation and verification have been rigorous and transparent in accordance with SAPs ECE.15 and the AV series of SAPs. This has allowed the impact to be assessed and future commitments and improvements to be explicitly recorded.
- The RP has made use of a sophisticated 3-D model in order to visualise the civil engineering design in submissions and meetings.
- Appropriate and rigorous quality assurance procedures have been demonstrated for the civil engineering design.

671. For civil engineering, as noted in Section 2.2.2, the structural design will be significantly influenced by the site-specific conditions that are excluded from the GDA scope. The GDA for civil engineering is therefore predicated on de-risking the main design concepts and methodologies via sample demonstrations of their application. Therefore, at GDA it is not realistic for the RP to demonstrate fully that risks are reduced ALARP. Rather the RP is expected to demonstrate that no significant issues remain that would preclude a full demonstration being made in the site-specific phase and that the overall design concept is viable. From my assessment recorded in the sections above, I judge that the RP has achieved this aim and adequately de-risked complex areas of the design methodologies. Therefore, I judge that the risks associated with this generic civil engineering design, at this stage of design development, have been reduced to ALARP.

4.13 Consolidation of Safety Case within PCSR Chapter 16 (Ref. 3)

672. During my assessment recorded in the sections above I have reviewed the consolidation of RQ responses within the RP's safety case submissions. I have noted some examples where I expected further consolidation of technical material by the RP. These areas are highlighted in the assessment text above.

673. The significant examples relate to the Internal Containment where aspects of the methodology and the validation work carried out by the RP's TSC have not been included in the formal safety case documentation. These areas are discussed in paragraphs 397, 420 and 427.

674. Overall, although improvements could be made, I am satisfied that the RP has subsumed the vast majority of the technical content of RQ responses and workshop material within the formally issued documentation. I judge that the matters highlighted above are not significant enough to undermine my judgement that the overall safety case consolidation process is adequate for civil engineering.

4.14 Comparison with Standards, Guidance and Relevant Good Practice

675. During my assessment recorded in the sections above, I have referenced the standards, guidance and relevant good practice used by the RP and my judgement of whether this accords with RGP. I have noted some examples where improvements or refinements could be made but I do not judge these to be significant. These areas are highlighted in the assessment text for the RP's attention, and where significant, captured via an assessment finding. Overall, for the purpose of GDA, I am satisfied that the RP has followed RGP as detailed in Section 2.4 and Annex 2.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

676. This report presents the findings of my civil engineering assessment of the generic UK HPR1000 design as part of the GDA process.

677. My assessment covered the following areas of the RP's civil engineering safety case:

- Output from previous GDA steps
- Regulatory Observations and Regulatory Queries
- Civil engineering safety case
- Design principles and methods for reinforced concrete primary structures
- Application of design principles and methods to:
  - Sample 1 – BFX (SSE1 structure on common raft)
  - Sample 2 – Internal containment
  - Sample 3 – Common raft foundation
  - Sample 4 – BNX and BDB/BDV (SSE1 structures on individual rafts)
  - Sample 5 – BEX (SSE2 structure)
  - Sample 6 – Malicious aircraft impact protection
- Further safety case considerations
- Demonstration that relevant risks have been reduced to ALARP
- Consolidation of safety case within PCSR Chapter 16

678. Based on my assessment, undertaken on a sampling basis, I have concluded the following:

- Residual matters in the form of *'Areas for Improvement'* and *'Open Points'* from the Civil Engineering GDA Step 3 assessment have been adequately resolved or are captured as assessment findings within this assessment.
- The Civil Engineering RO raised in GDA Step 2, RO-UKHPR1000-0009, has been adequately resolved and closed. Civil engineering input to other RO's as outlined in Table 5 has been provided and these RO's have been adequately resolved and closed.
- Regarding the civil engineering safety case, I am satisfied that the overall structure, scope and limitations are appropriate for the purpose of GDA, and that the cross-cutting inputs are predominantly coherent. Further, I am satisfied with the traceability and clarity of the safety functions and the RP's use of SFR schedules. I judge that the RP has developed the civil engineering safety case to a proportionate level that meets the purpose of GDA. I consider that this provides an adequate reference point from which to develop the detailed civil engineering design more fully in the site-specific phase.
- I am content that the design principles and methods articulated by the RP are appropriate for the purposes of GDA and are adequately aligned with RGP and the intent of the ONR SAPs. These methodologies provide a robust baseline ready for future augmentation to include further detail and site-specific aspects.
- From my assessment of the application of the design principles and methodologies to the 6 sample areas, I am content that the RP has presented an adequate demonstration of the application of their methodologies that has fulfilled the purpose of GDA. I make the following specific points:
  - Sample 1 – BFX: I am satisfied that the RP's design basis analysis for the BFX adequately meets RGP and the intent of SAP ECE.13. I am satisfied that the RPs beyond design basis evaluation indicates that adequate margins are available to satisfy SAPs ECE.1 and ECE.2 (specifically SAPs paragraphs 334 and 337). I note that the ongoing BFX design modifications

will necessitate a complete re-analysis of the structure for the revised geometry in the site-specific phase.

- Sample 2 – Internal containment: I am satisfied for GDA that the design basis analysis for the internal containment adequately meets RGP and the intent of SAP ECE.13. I am satisfied that the beyond design basis margins established by the ultimate capacity evaluation are sufficient to satisfy SAPs ECE.1 and ECE.2 (SAP paragraphs 334 and 337) and that severe accident scenarios are appropriately considered. With respect to the probabilistic evaluation, I consider the fragility curves to be adequate for GDA purposes.
- Sample 3 – Common raft foundation: Due to the site-specific nature of any foundation design, combined with the very soft characteristics of the 'Target Site', further development and optimisation of the common raft design will be needed in the site-specific phase. Nonetheless, based on the credible solutions developed by the RP, I judge that the GDA design is deployable given appropriate levels of site-specific design optimisation.
- Sample 4 – BNX and BDB/BDV: During my assessment of the SSI and SSSI for the BNX I have gained sufficient assurance in the RP's methodologies for structures of this type and classification. Subject to a more complete analysis methodology being developed, I consider the approach appropriate for the site-specific phase. Furthermore, for the BDB/BDV buildings, I consider that further work is required in the detailed design phase to ensure an appropriate methodology for non-standard fires is established. Overall, I am satisfied with the differences in design methodologies for SSE1 structures not on the common raft foundation.
- Sample 5 – BEX: I am satisfied with the differences in design methodologies for SSE2 structures not on the common raft foundation.
- Sample 6 – Malicious aircraft impact protection: I am satisfied that the GDA design is robust against aircraft impact and that this can be further optimised during the site-specific phase.

- From my assessment of the aspects of novelty, radiation protection, defence in depth, constructability, EIMT and decommissioning, I conclude that these have been adequately considered by the RP and meet the purposes of GDA.
- With respect to demonstrating that risks have been reduced ALARP, I am satisfied the RP has adequately demonstrated the application of RGP. Furthermore, I am content that the RP has demonstrated that no significant issues remain that would preclude the full site-specific realisation of the design, demonstrating risks are reduced ALARP.
- From my review of the consolidated safety case, I am satisfied that the RP has subsumed the technical content of RQ responses within the formally issued documentation and consider the safety case consolidation process to be adequate.

679. I have raised assessment findings to cover areas for further design development and optimisation for the detailed and site-specific design phases. I judge that these do not challenge the overall conclusions above.

680. Overall, based on my sample assessment of the safety case for the generic UK HPR1000 design undertaken in accordance with ONR's procedures, I am satisfied that the case presented within the PCSR and supporting documentation is adequate. On this basis, I am content that a Design Acceptance Confirmation (DAC) should be granted for the generic UK HPR1000 design from a civil engineering perspective.

5.2 Recommendations

681. Based upon my assessment detailed in this report, I recommend that:

- **Recommendation 1**: From a civil engineering perspective, ONR should grant a DAC for the generic UK HPR1000 design.
- **Recommendation 2**: The 22 Assessment Findings identified in this report should be resolved by the licensee for a site-specific application of the generic UK HPR1000 design.

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113. *UK HPR1000 Step 4 Management for Safety and Quality Assurance Assessment Report,* ONR-NR-AR-21-003, Rev. 0, January 2022, ONR. [CM9 Ref. 2021/42541]
114. *UKHPR1000 GDA Step 4 Modifications Log,* , ONR. [CM9 Ref. 2020/313979]
115. *General requirements of protection design against internal and external hazards,* GHX00100028DOZJ03GN, Rev F., January 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/8467]

116. *External hazards schedule report,* GHX86000015DOZJ03GN, Rev. G, May 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/43513]
117. *Internal hazards schedule report,* GHX84200051DOZJ03GN, Rev. C, April 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/30052]
118. *UK HPR1000 Fault Schedule,* GHX00600276DRAF02GN, Rev. E, August 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/64934]
119. *Pre-Construction Safety Report: Chapter 20: MSQA and Safety Case Management,* HPR/GDA/PCSR/0020, Rev. 002, November 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/85134]
120. *Pre-Construction Safety Report: Chapter 12: Design Basis Condition Analysis,* HPR/GDA/PCSR/0012, Rev. 002, November 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/85152]
121. *Pre-Construction Safety Report: Chapter 13: Design Extension Conditions and Severe Accident Analysis,* HPR/GDA/PCSR/0013, Rev. 002, November 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/85150]
122. *Combined Hazards Safety Assessment Report,* GHX84200031DOZJ03GN, Rev. A, September 2020, General Nuclear Systems Ltd.. [CM9 Ref. 2020/288805]
123. *External hazards combination safety evaluation report,* GHX86000001DOZJ00GN, Rev. B, December 2020, General Nuclear Systems Ltd.. [CM9 Ref. 2020/322027]
124. *Beyond Design Basis External Hazards Evaluation Methodology,* GHX00100003DOZJ00GN, Rev. B, July 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/58569]
125. *Cliff edge effect of extreme environmental hazard for civil engineering structure,* GHXNIX10006DWJG42GN, Rev. B, April 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/35807]
126. *Malicious Aircraft Impact - UK Expectations,* December 2013, ONR. [CM9 Ref. 2013/471315]
127. *Civil engineering schedule report,* GHXNIX10058DWJG42GN, Rev. B, July 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/56418]
128. *UK HPR1000 Step 4 Mechanical Engineering Assessment Report,* ONR-NR-AR-21-004, Rev.0, January 2022, ONR. [CM9 Ref. 2021/53696]
129. *Basis of safety case for aircraft impact,* GHXNIX10021DWJG42GN, Rev. H, June 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/50512]
130. *Dropped Loads Safety Assessment Report for Fuel Building,* GHX84200048DOZJ03GN, Rev. A, October 2020, General Nuclear Systems Ltd.. [CM9 Ref. 2020/304800]
131. *UK HPR1000 Step 4 Conventional Health and Safety Assessment Report,* ONR-NR-AR-21-009, Rev. 0, January 2022, ONR. [CM9 Ref. 2021/50011]
132. *Pre-Construction Safety Report: Chapter 10: Auxiliary Systems,* HPR/GDA/PCSR/0010, Rev. 002, November 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/85154]
133. *Evaluation of internal containment under tendon failure,* GHXRIX10008DWJG42GN, Rev. B, April 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/36839]
134. *Reactor Building Fuel Transfer Canal Shielding Design report,* GHX00100048DNFP03GN, Rev. B, January 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/4828]
135. *GDA Step 4 - EPR PSA Support: Containment Over-pressurisation Fragility,* 2120812-R-05, 1, May 2011, ABS Consulting. [CM9 Ref. 2013/38908]
136. *Level 2 PSA,* GHX00650140DOZJ02GN, Rev. C, July 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/51693]
137. *OFFICIAL SENSITIVE (SNI) - Level 4 Meeting to Discuss Malicious Aircraft Impact Threat Definitions,* ONR-NR-CR-19-288, Rev. 0, October 2019, ONR. [CM9 Ref. 2019/309029]

138. *Aircraft impact evaluation report,* GHXNIX10023DWJG42GN, Rev. E, March 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/21870]
139. *Aircraft impact dynamic analysis report,* GHXNIX10025DWJG42GN, Rev. C, March 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/21875]
140. *Fuel Building Shielding Design Report,* GHX00100033DNFP03GN, Rev. E, April 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/36843]
141. *Consistency evaluation for design of facilitating decommissioning,* GHX71500005DNFF03GN, Rev. E, March 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/28168]
142. *UK HPR1000 Step 4 Decommissioning Assessment Report,* ONR-NR-AR-21-015, Rev. 0, January 2022, ONR. [CM9 Ref. 2021/51328]
143. *Decommissioning waste management proposal,* GHX71500009DNFF03GN, Rev. G, July 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/54418]
144. *The design requirements for facilitating decommissioning,* GHX71500016DNFF03GN, C, April 2020, General Nuclear Systems Ltd.. [CM9 Ref. 2020/131216]
145. *Decontamination processes and techniques during decommissioning,* GHX71500010DNFF03GN, Rev. D, March 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/28169]
146. *Preliminary Decommissioning Plan Report,* GHX71500004DNFF03GN, Rev. H, July 2021, General Nuclear Systems Ltd.. [CM9 Ref. 2021/58813]
147. *Seismic analysis report for structure,* GHXNIX10014DWJG42GN, Rev. A, April 2019, General Nuclear Systems Ltd.. [CM9 Ref. 2019/122796]
148. *Structural analysis and design report,* GHXNIX10015DWJG42GN, Rev. A, April 2019, General Nuclear Systems Ltd.. [CM9 Ref. 2019/122517]
149. *Scope for UK HPR1000 GDA Project,* HPR/GDA/REPO/0007, Rev. 001, July 2019, General Nuclear Systems Ltd.. [CM9 Ref. 2019/209339]
150. *Methodology of safety categorisation and classification,* GHX00100062DOZJ03GN, Rev. B, June 2018, General Nuclear Systems Ltd.. [CM9 Ref. 2018/199731]
151. *ALARP Methodology,* GHX00100051DOZJ03GN, Rev. D, May 2020, General Nuclear Systems Ltd.. [CM9 Ref. 2020/139106]
152. *Aircraft impact analysis load time function report,* GDA-REC-MOT-SEC-000022, Rev. 2, March 2020, General Nuclear Systems Ltd.. [CM9 Ref. 2020/84081]
153. *Identification and management of GDA issues, assessment findings and minor shortfalls for the GDA of UK HPR1000,* ONR-GEN-IN-021, Rev. 0, May 2020, ONR. [CM9 Ref. 2019/218899]
154. *A Guide to the Regulatory Process,* Rev. 0, September 2013, ONR. http://www.onr.org.uk/newreactors/guidance-assessment.htm
155. *Generic Design Assessment Technical Guidance,* ONR-GDA-GD-007, Rev. 0, May 2019, ONR. https://www.onr.org.uk/new-reactors/guidance-assessment.htm
156. *Generic Design Assessment Guidance to Requesting Parties,* ONR-GDA-GD-006, Rev. 0, October 2019, ONR. https://www.onr.org.uk/new-reactors/onr-gda-gd-006.pdf
157. *Guidance on Production of Reports for Permissioning,* ONR-NS-PER-GD-015, Rev. 2, June 2020, ONR. [CM9 Ref. 2020/288716]
158. *ONR Editorial and Formatting Style Guide,* ONR-COM-GD-002, Rev. 4, May 2018, ONR. [CM9 Ref. 2018/162407]
159. *UK HPR1000 Aircraft impact gap analysis report,* GDA-REC-MOT-SEC-000001, Rev. 4, December 2019, General Nuclear Systems Ltd.. [CM9 Ref. 2020/4202]

Annex 1

Relevant Safety Assessment Principles Considered During the Assessment

SAP No.	SAP Title	Description
SC.1	The regulatory assessment of safety cases: Safety case production process	The process for producing safety cases should be designed and operated commensurate with the hazard, using concepts applied to high reliability engineered systems.
SC.2	The regulatory assessment of safety cases: Safety case process outputs	The safety case process should produce safety cases that facilitate safe operation.
SC.3	The regulatory assessment of safety cases: Lifecycle aspects	For each lifecycle stage, control of the hazard should be demonstrated by a valid safety case that takes into account the implications from previous stages and for future stages.
SC.4	The regulatory assessment of safety cases: Safety case characteristics	A safety case should be accurate, objective and demonstrably complete for its intended purpose.
SC.5	The regulatory assessment of safety cases: Optimism, uncertainty and conservatism	Safety cases should identify areas of optimism and uncertainty, together with their significance, in addition to strengths and any claimed conservatism.
SC.7	The regulatory assessment of safety cases: Safety case maintenance	A safety case should be actively maintained throughout each of the lifecycle stages and reviewed regularly.
SC.8	The regulatory assessment of safety cases: Safety case ownership	Ownership of the safety case should reside within the dutyholder's organisation with those who have direct responsibility for safety.
EKP.1	Engineering principles: key principles, (Inherent safety)	The underpinning safety aim for any nuclear facility should be an inherently safe design, consistent with the operational purposes of the facility.
EKP.2	Engineering principles: key principles, (Fault tolerance)	The sensitivity of the facility to potential faults should be minimised.
EKP.3	Engineering principles: key principles, (Defence in depth)	Nuclear facilities should be designed and operated so that defence in depth against potentially significant faults or failures is achieved by the provision of multiple independent barriers to fault progression.

SAP No.	SAP Title	Description
ECS.1	Engineering principles: safety classification and standards, (Safety categorisation)	The safety functions to be delivered within the facility, both during normal operation and in the event of a fault or accident, should be identified and then categorised based on their significance with regard to safety.
ECS.2	Engineering principles: safety classification and standards, (Safety classification of structures, systems and components)	Structures, systems and components that have to deliver safety functions should be identified and classified on the basis of those functions and their significance to safety.
ECS.3	Engineering principles: safety classification and standards, (Codes and standards)	Structures, systems and components that are important to safety should be designed, manufactured, constructed, installed, commissioned, quality assured, maintained, tested and inspected to the appropriate codes and standards.
ERL.4	Engineering principles: Reliability claims (Margins of conservatism)	Where safety-related systems and/or other means are claimed to reduce the frequency of a fault sequence, the safety case should include a margin of conservatism to allow for uncertainties
EHA.4	Engineering principles: External and Internal hazards (Frequency of Initiating event)	For natural external hazards, characterised by frequency of exceedance hazard curves and internal hazards, the design basis event for an internal or external hazard should be derived to have a predicted frequency of exceedance that accords with Fault Analysis Safety Assessment Principle FA.5. The thresholds set in Principle FA.5 for design basis events are 1 in 10 000 years for external hazards and 1 in 100 000 years for man-made external hazards and all internal hazards (see also SAP paragraph 629)
EHA.6	Engineering principles: External and Internal hazards, (Analysis)	The effects of internal and external hazards that could affect the safety of the facility should be analysed. The analysis should take into account hazard combinations, simultaneous effects, common cause failures, defence in depth and consequential effects.
EHA.7	Engineering principles: External and Internal hazards, (Cliff-edge effects)	A small change in design basis fault or event assumptions should not lead to a disproportionate increase in radiological consequences.
EHA.18	Engineering principles: External and Internal hazards, (Beyond Design Basis Events)	Fault sequences initiated by internal and external hazards beyond the design basis should be analysed applying an appropriate combination of engineering, deterministic and probabilistic assessments.

SAP No.	SAP Title	Description
ELO.1	Engineering principles: layout, (Access)	The design and layout should facilitate access for necessary activities and minimise adverse interactions while not compromising security aspects.
ELO.4	Engineering principles: layout, (Minimisation of the effects of incidents)	The design and layout of the site, its facilities (including enclosed plant), support facilities and services should be such that the effects of faults and accidents are minimised.
FA.5	Fault Analysis: Design basis analysis (Initiating faults)	The safety case should list all initiating faults that are included within the design basis analysis of the facility
AV.1	Fault Analysis: assurance of validity of data and models, (Theoretical Models)	Theoretical models should adequately represent the facility and site.
AV.2	Fault Analysis: assurance of validity of data and models, (Calculation Methods)	Calculation methods used for the analyses should adequately represent the physical and chemical processes taking place.
AV.4	Fault Analysis: assurance of validity of data and models, (Computer Models)	Computer models and datasets used in support of the safety analysis should be developed, maintained and applied in accordance with quality management procedures.
AV.5	Fault Analysis: assurance of validity of data and models, (Documentation)	Documentation should be provided to facilitate review of the adequacy of the analytical models and data.
AV.6	Fault Analysis: assurance of validity of data and models, (Sensitivity Studies)	Studies should be carried out to determine the sensitivity of the analysis (and the conclusions drawn from it) to the assumptions made, the data used and the methods of calculation.
EAD.1	Engineering principles: ageing and degradation, (Safe working life)	The safe working life of structures, systems and components that are important to safety should be evaluated and defined at the design stage.
EAD.2	Engineering principles: ageing and degradation, (Lifetime margins)	Adequate margins should exist throughout the life of a facility to allow for the effects of materials ageing and degradation processes on structures, systems and components.

SAP No.	SAP Title	Description
ECE.1	Engineering principles: civil engineering, (Functional Performance)	The required safety functions and structural performance of the civil engineering structures under normal operating, fault and accident conditions should be specified.
ECE.2	Engineering principles: civil engineering, (Independent Arguments)	For structures requiring the highest levels of reliability, multiple, independent and diverse arguments should be provided in the safety case.
ECE.3	Engineering principles: civil engineering, (Defects)	It should be demonstrated that structures important to safety are sufficiently free of defects so that their safety functions are not compromised, that identified defects can be tolerated, and that the existence of defects that could compromise safety functions can be established through their lifecycle.
ECE.6	Engineering principles: civil engineering: design, (Loadings)	Load development and a schedule of load combinations, together with their frequencies, should be used as the basis for structural design. Loadings during normal operating, testing, design basis fault and accident conditions should be included.
ECE.7	Engineering principles: civil engineering: design, (Foundations)	The foundations and sub-surface structures should be designed to meet their safety functions requirements specified for normal operation and fault conditions with an absence of cliff edge effects beyond the design basis.
ECE.8	Engineering principles: civil engineering: design, (Inspectability)	Designs should allow key load bearing elements to be inspected and, where necessary, maintained.
ECE.10	Engineering principles: civil engineering: design, (Groundwater)	The design should be such that the facility remains stable against possible changes in the groundwater conditions.
ECE.12	Engineering principles: civil engineering: structural Analysis and Model Testing, (Structural Analysis and Model Testing)	Structural analysis and/or model testing should be carried out to support the design and should demonstrate that the structure can fulfil its safety functional requirements over the full range of loading for the lifetime of the facility.
ECE.13	Engineering principles: civil engineering: structural Analysis and Model Testing, (Use of Data)	The data used in structural analysis should be selected or applied so that the analysis is demonstrably conservative.
ECE.14	Engineering principles: civil engineering: structural Analysis and Model Testing, (Sensitivity studies)	Studies should be carried out to determine the sensitivity of analytical results to the assumptions made, the data used, and the methods of calculation.

SAP No.	SAP Title	Description
ECE.15	Engineering principles: civil engineering: structural Analysis and Model Testing, (Validation of Methods)	Where analysis has been carried out on civil structures to derive static and dynamic structural loadings for the design, the methods used should be adequately validated and the data verified.
ECE.16	Engineering principles: civil engineering: construction, (Materials)	The construction materials used should comply with the design methodologies employed and be shown to be suitable for enabling the design to be constructed and then operated, inspected and maintained throughout the life of the facility.
ECE.17	Engineering principles: civil engineering: construction, (Prevention of Defects)	The construction should use appropriate materials, proven techniques and a quality management system to minimise defects that might affect the required integrity of structures.
ECE.18	Engineering principles: civil engineering: construction, (Inspection during construction)	Provision should be made for inspection and testing during construction to demonstrate that appropriate standards of workmanship etc. have been achieved.
ECE.20	Engineering principles: civil engineering: in-service inspection and testing, (Inspection, testing and monitoring)	Provision should be made for inspection testing and monitoring during normal operations aimed at demonstrating that the structure continues to meet its safety functional requirements. Due account should be taken of the periodicity of the activities.
ECE.21	Engineering principles: civil engineering: in-service inspection and testing, (Proof pressure tests)	Pre-stressed concrete pressure vessels and containment structures should be subjected to a proof pressure test, which may be repeated during the life of the facility
ECE.22	Engineering principles: civil engineering: in-service inspection and testing, (Leak tightness)	Civil engineering structures that retain or prevent leakage should be tested for leak tightness prior to operation.
ECE.24	Engineering principles: civil engineering: in-service inspection and testing, (Settlement)	There should be arrangements to monitor civil engineering structures during and after construction to check the validity of predictions of performance made during the design and for feedback into design reviews
ECE.25	Engineering principles: civil engineering: design, (Provision for Construction)	Items important to safety should be designed so that they can be manufactured, constructed, assembled, installed and erected in accordance with established processes that ensure the achievement of the design specifications and the required level of safety. The effects of construction hazards on any nearby safety related SSCs should be taken into account.

SAP No.	SAP Title	Description
ECE.26	Engineering principles: civil engineering: design, (Provision for Decomissioning)	Special consideration should be given at the design stage to the incorporation of features to facilitate radioactive waste management and the future decommissioning and dismantling of the facility.

Annex 2

International Guidance, Codes and Standards relevant to this assessment

Text Reference	Codes and Standards
ACI318-08	ACI, Building Code Requirements for Structural Concrete, ACI318-08, 2008.
ACI318-19	ACI, Building Code Requirements for Structural Concrete, ACI318-19, 2019.
ACI349-13	ACI, Code Requirements for Nuclear Safety-Related Concrete Structures, ACI349-13, 2013.
ACI349.1R-07	ACI, Reinforced Concrete Design for Thermal Effects on Nuclear Power Plant Structures, ACI349.1R-07, 2007.
ACI350.3-06	ACI, Seismic Design of Liquid-Containing Concrete Structures and Commentary, ACI350.3-06, 2006.
ACI359-17	ACI-ASME Joint Technical Committee, Code for Concrete Containments, ACI359-17, 2017.
AISC N690-18	AISC, Specification for Safety-Related Steel Structures for Nuclear Facilities, AISC N690-18, 2018.
ASCE4-16	ASCE, Seismic Analysis of Safety-Related Nuclear Structures, ASCE4-16, 2017.
ASCE43-05	ASCE, Seismic design criteria for structures, systems, and components in nuclear facilities, ASCE43-05, 2005.
ASCE43-19	ASCE, Seismic design criteria for structures, systems, and components in nuclear facilities, ASCE43-19, 2019.
BS4449	BSI, Steel for the reinforcement of concrete, BS4449:2005, 2005.
BS8500-1	BSI, Concrete – complementary British Standard to BS EN 206. Method of specifying and guidance for the specifier, BS8500-1:2015, 2015.
BS8666	BSI, Scheduling, dimensioning, bending and cutting of steel reinforcement for concrete. Specification, BS8666:2005, September 2005.
C766	CIRIA, Control of cracking caused by restrained deformation in concrete, CIRIA C766, 2018.
EN1992-1-1	BSI, Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings, BS EN 1992-1-1:2004, 2004.
EN1992-1-2	BSI, Eurocode 2: Design of concrete structures – Part 1-2: General rules – Structural fire design, BS EN 1992-1-2:2004, 2004.

Text Reference	Codes and Standards
EN1992-2	BSI, Eurocode 2: Design of concrete structures – Part 2: Concrete bridges – Design and detailing rules, BS EN 1992-2:2005, 2005.
EN1992-3	BSI, Eurocode 2: Design of concrete structures – Part 3: Liquid retaining and containment structures, BS EN 1992-3:2006, 2006.
EPRI NP-6041-SL	EPRI, A Methodology for Assessment of Nuclear Power Plant Seismic Margin, EPRI NP-6041-SL, August 1991.
ETC-C	AFCEN, EPR Technical Code for Civil Works with additions and amendments for the United Kingdom, ETC-C – 2010 Edition + UK Companion Document, ETC-C, 2010.
EUR	EUR, European Utility Requirements for LWR Nuclear Power Plants, EUR, 2017.
fib MC2010	fib, Model Code for Concrete Structures 2010, 2013. Specifically for fire design, bulletin N° 46. “Fire design of concrete structures - structural behaviour and assessment”. (ISBN 978-2-88394-086-4, July 2008)
IAEA SF-1	Fundamental Safety Principles. IAEA Safety Fundamentals No SF-1. November 2006
IAEA SSR-2/1	IAEA, Safety of Nuclear Power Plants: Design, SSR-2/1, Rev. 1, 2016.
IAEA SSG-53	IAEA, Design of the Reactor Containment and Associated Systems for Nuclear Power Plants, SSG-53, 2019.
IAEA NS-G-1.5	IAEA, External Events Excluding Earthquakes in the Design of Nuclear Power Plants, NS-G-1.5, 2003.
IAEA SRS86	IAEA, Safety Aspects of Nuclear Power Plants in Human Induced External Events: General Considerations, Safety Reports Series No. 86, 2017.
IAEA SRS87	IAEA, Safety Aspects of Nuclear Power Plants in Human Induced External Events: Structural Assessment, Safety Reports Series No. 87, 2018.
IAEA SRS88	IAEA, Safety Aspects of Nuclear Power Plants in Human Induced External Events: Margin Assessment, Safety Reports Series No. 88, 2018.
ISO834	ISO 834 Fire Resistance Tests – Elements of Building Construction – Parts 1:14, International Organisation for Standardisation, 1999-2019.
NEI 07-13_8P	NEI, Methodology for Performing Aircraft Impact Assessments for New Plant Designs, NEI 07-13, Rev. 8P, April 2011.
NUREG-0800	USNRC, Standard Review Plan, Regulatory Guide 0800, Rev. 3, May 2010.

Text Reference	Codes and Standards
NUREG 1.216	USNRC, Containment structural integrity evaluation for internal pressure loadings above design basis pressure, Regulatory Guide 1.216, Rev. 0, August 2010.
NUREG/CR-6906	USNRC, Containment Integrity Research at Sandia National Laboratory - An Overview, NUREG/CR-6906, July 2006.
prEN 10138-1	CEN, Prestressing steels – Part 1: General requirements, prEN 10138-1, 2000.
RCC-CW	AFCEN, Rules for design and construction of PWR nuclear civil works, RCC-CW, 2016.

Annex 3

RQ's raised during the civil engineering assessment

RQ Number	RQ Title	Link to GDA Step 3 assessment
RQ-UKHPR1000-0579	Design methods for structures of different classification	AFI-1
RQ-UKHPR1000-0580	Demonstration of safety case for barriers against internal hazards	AFI-2, AFI-4
RQ-UKHPR1000-0581	Demonstration methods for adequacy of spatial design	AFI-5
RQ-UKHPR1000-0582	Demonstration of beyond design basis provision	AFI-6
RQ-UKHPR1000-0583	Fire strategy for aircraft impact	AFI-7
RQ-UKHPR1000-0584	Implementation of damping in accordance with ASCE4-16	AFI-8
RQ-UKHPR1000-0585	Structure-soil-structure interaction	AFI-9
RQ-UKHPR1000-0586	Design calculations	AFI-10 & AFI-20
RQ-UKHPR1000-0587	Handling of stresses arising from thermal actions	AFI-11
RQ-UKHPR1000-0588	Modelling of internal containment	AFI-12
RQ-UKHPR1000-0589	Reporting the design basis	AFI-13 & AFI-14
RQ-UKHPR1000-0590	Reporting the design basis loads	AFI-15 & OP-29
RQ-UKHPR1000-0591	Structural response to aircraft impact induced vibration	AFI-16 & AFI-18
RQ-UKHPR1000-0592	Structural strain limits	AFI-17
RQ-UKHPR1000-0593	Exclusions from GDA	AFI-19
RQ-UKHPR1000-0594	Integration of new hazard data into civil engineering	OP-1
RQ-UKHPR1000-0595	Provision of technical drawings	OP-2 & OP-11
RQ-UKHPR1000-0596	Layout	OP-6
RQ-UKHPR1000-0597	Design of SSCs for actions from equipment	OP-9
RQ-UKHPR1000-0598	Averaging of element results for design calculations	OP-10
RQ-UKHPR1000-0599	Modelling of pre-stressing tendons	OP-12
RQ-UKHPR1000-0600	Modelling of the gusset area	OP-14 & OP-16
RQ-UKHPR1000-0601	Sliding resistance load paths for the internal structures	OP-15
RQ-UKHPR1000-0602	Use of existing ABAQUS model for verification	OP-17
RQ-UKHPR1000-0603	Fire design methodology for non-standard fires	OP-18
RQ-UKHPR1000-0604	Demonstration of safety case golden thread for air tightness	OP-22
RQ-UKHPR1000-0605	Justification for grouted post tensioning system	OP-23
RQ-UKHPR1000-0606	Sliding resistance load paths for the internal structures	OP-27
RQ-UKHPR1000-0610	Queries on the draft BFX safety functional requirement schedule	AFI-3, OP-21
RQ-UKHPR1000-0616	Queries on the general requirements for design of the inner containment liner report	

RQ Number	RQ Title	Link to GDA Step 3 assessment
RQ-UKHPR1000-0747	EUR Spectra: Definition, Use and Control Point	
RQ-UKHPR1000-0752	Exhaustivity of Safety Functional Requirements at GDA	
RQ-UKHPR1000-0753	Traceability of fault conditions into design basis conditions for civil engineering	
RQ-UKHPR1000-0754	Incompatibility of PCSR chapter 16 SFR summary tables with SFR schedule for BFX	
RQ-UKHPR1000-0755	Correctness of civil engineering document hierarchy and its implementation	
RQ-UKHPR1000-0756	Application of seismic load and modelling of equipment for seismic analysis	
RQ-UKHPR1000-0757	Description of SSI modelling approach and narrative on SSI results	
RQ-UKHPR1000-0758	Time history development	
RQ-UKHPR1000-0759	Basis for determining structures where SSSI analysis is required	
RQ-UKHPR1000-0760	Scope of planned sensitivity and V&V studies related to seismic analysis at GDA	
RQ-UKHPR1000-0761	Process for transferring analysis results between analysis models including methods for validation	
RQ-UKHPR1000-0762	Use of local models for design substantiation of key civil engineering structures	
RQ-UKHPR1000-0763	Walk through of analysis and design process for common raft	
RQ-UKHPR1000-0764	Justification of geotechnical parameters used for GDA	
RQ-UKHPR1000-0765	Design substantiation of common raft at GDA	
RQ-UKHPR1000-0766	Clarification of information pertaining to the common raft design	
RQ-UKHPR1000-0767	Common raft golden thread	
RQ-UKHPR1000-0768	Analysis methodology for design of liner structures (not including IC liner)	
RQ-UKHPR1000-0769	Methodology for considering BFX crane loads in static and seismic analysis	
RQ-UKHPR1000-0770	Clarification on proposed changes to durability requirements	
RQ-UKHPR1000-0771	Inclusion of impulsive and convective effects for hydrodynamic pool loads	
RQ-UKHPR1000-0772	Safety classification for cranes	
RQ-UKHPR1000-0773	Analysis and methodology for the assessment of SFP thermal loads	
RQ-UKHPR1000-0782	Containment Ultimate Capacity Acceptance Criteria	
RQ-UKHPR1000-0789	Aircraft impact on buildings containing radioactive material	
RQ-UKHPR1000-0790	Aircraft impact fire spread rules	
RQ-UKHPR1000-0791	Examples of aircraft impact fire spread analysis	
RQ-UKHPR1000-0792	EPW dampers	
RQ-UKHPR1000-0805	Design Formulae for reinforced concrete 2D shell elements	

RQ Number	RQ Title	Link to GDA Step 3 assessment
RQ-UKHPR1000-0816	Comments on proposed V&V and sensitivity studies	
RQ-UKHPR1000-0842	Hazard Combinations	
RQ-UKHPR1000-0852	Fault schedule information flowing into Civil Engineering	
RQ-UKHPR1000-0853	General queries related to Basis of Safety Case for Internal and External Containment	
RQ-UKHPR1000-0854	Applicability of LLSFs for Civil Engineering and declaration of safety functions in SFR schedule	
RQ-UKHPR1000-0855	Clarification of scope for cliff edge and beyond design basis assessments	
RQ-UKHPR1000-0856	Internal containment seismic analysis	
RQ-UKHPR1000-0857	Design methodology for Internal Containment liner	
RQ-UKHPR1000-0858	Design methodology for polar crane bracket	
RQ-UKHPR1000-0859	Internal Containment construction photos for FCG3	
RQ-UKHPR1000-0860	PDMS modelling	
RQ-UKHPR1000-0861	Internal Containment liner material properties	
RQ-UKHPR1000-0862	Internal Containment liner performance requirements	
RQ-UKHPR1000-0867	Further queries on seismic analysis of gusset region of Internal Containment	
RQ-UKHPR1000-0868	Interface between Internal Containment and Common Raft analysis models	
RQ-UKHPR1000-0869	Approach for consideration of tendon failure in Internal Containment	
RQ-UKHPR1000-0873	Design requirements for doors and similar features	
RQ-UKHPR1000-0878	Modelling approach for Internal Containment	
RQ-UKHPR1000-0879	Alternative design approach for non-standard zone of Internal Containment	
RQ-UKHPR1000-0880	Validation & Verification studies for Internal Containment	
RQ-UKHPR1000-0881	Design margin and presentation of results for Internal Containment	
RQ-UKHPR1000-0888	General queries on loading for Internal Containment design	
RQ-UKHPR1000-0889	Post-tensioning load calculation and modelling for Internal Containment	
RQ-UKHPR1000-0890	Thermal analysis methodology for Internal Containment design	
RQ-UKHPR1000-0891	Further queries on structural analysis under thermal loads of gusset region of Internal Containment	
RQ-UKHPR1000-0896	Use of REINCAL for Internal Containment design	
RQ-UKHPR1000-0897	Determination of appropriate loads for Internal Containment (IC) and IC Liner and their application for DBA and BDBA	
RQ-UKHPR1000-0902	Aircraft Selected for Impact Locations	
RQ-UKHPR1000-0903	Fuel Ingress Through Cracks	
RQ-UKHPR1000-0904	Evaluation Report Updates	

RQ Number	RQ Title	Link to GDA Step 3 assessment
RQ-UKHPR1000-0905	Impact Locations for Global Stability Analysis	
RQ-UKHPR1000-0906	Support Systems Required for Safe Shutdown	
RQ-UKHPR1000-0908	Query on common cause failures resulting from aircraft impact	
RQ-UKHPR1000-0909	Additional queries on fire spread through openings	
RQ-UKHPR1000-0910	Follow up queries related to definition and use of EUR spectra	
RQ-UKHPR1000-0927	Static Geotechnical Parameters	
RQ-UKHPR1000-0928	Common raft thickness	
RQ-UKHPR1000-0938	Design of concrete elements for out-of-plane shear	
RQ-UKHPR1000-0939	Averaging assumptions for in-plane shear	
RQ-UKHPR1000-0940	Averaging out-of-plane shear	
RQ-UKHPR1000-0941	Bounding Internal Hazard loads used for Civil Engineering design	
RQ-UKHPR1000-0942	Concrete crushing check under the vector resultant of Vx and Vy	
RQ-UKHPR1000-0943	Design and analysis of structural elements subjected to Internal Hazard loads	
RQ-UKHPR1000-0944	SFRs for liner within Basis of Safety Case for BFX	
RQ-UKHPR1000-0945	Containment Function of BFX Structure	
RQ-UKHPR1000-0947	Local Model 2 (LM-2) small penetration validation	
RQ-UKHPR1000-0948	Local Model 2 (LM-2) pipe load application	
RQ-UKHPR1000-0949	Use and validation of Local Model 3 (LM-3)	
RQ-UKHPR1000-0950	Comparison of Impact-induced Response Spectra from Different Aircraft Types	
RQ-UKHPR1000-0951	Aircraft Impact Locations for Dynamic Analysis	
RQ-UKHPR1000-0977	Effects of Aircraft Impact Induced Explosions on Unprotected SSCs	
RQ-UKHPR1000-0978	Shielding Analysis	
RQ-UKHPR1000-0979	Effects of Cranes Falling Due to Aircraft Impact	
RQ-UKHPR1000-1004	Aircraft Impact Perforation Checks	
RQ-UKHPR1000-1005	Treatment of SFRs relating to the Common Raft	
RQ-UKHPR1000-1006	Detailed questions on Basis of Safety Case reports for BFX, BRX Internal Structures, BNX and BEX	
RQ-UKHPR1000-1007	PDMS, design definition and change control at GDA	
RQ-UKHPR1000-1019	Thermal reduction factors	
RQ-UKHPR1000-1020	Thermal analysis feeding into structural design	
RQ-UKHPR1000-1040	Inclination and differential settlements	
RQ-UKHPR1000-1041	BNX external hazards	
RQ-UKHPR1000-1042	BNX Seismic Analysis – Validation of Floor Response Spectra	
RQ-UKHPR1000-1043	Seismic analysis of BEX	

RQ Number	RQ Title	Link to GDA Step 3 assessment
RQ-UKHPR1000-1044	Seismic joint assessment	
RQ-UKHPR1000-1045	Seismic detailing within design process	
RQ-UKHPR1000-1046	Analysis of spent fuel pool under Fuel Assembly drop	
RQ-UKHPR1000-1082	Temperature Fields for Internal Containment and Common Raft Foundation	
RQ-UKHPR1000-1123	Civil engineering analysis of Internal Hazard loads for non-barrier structural elements	
RQ-UKHPR1000-1156	Validation of Internal Containment mesh	
RQ-UKHPR1000-1157	Validation of tendon modelling within IC	
RQ-UKHPR1000-1158	Validation of discontinuities within IC	
RQ-UKHPR1000-1159	Enveloping seismic timestep results	
RQ-UKHPR1000-1160	Validation of soil spring stiffnesses	
RQ-UKHPR1000-1161	Validation of boundary conditions for IC	
RQ-UKHPR1000-1162	Annulus temperature in thermal analysis of IC	
RQ-UKHPR1000-1271	Polar Crane Bracket Loading	
RQ-UKHPR1000-1272	Load on IC from liner under DBA	
RQ-UKHPR1000-1273	Boundary Conditions of IC liner local model	
RQ-UKHPR1000-1274	Failure sequence of IC liner components	
RQ-UKHPR1000-1275	BNX raft foundation	
RQ-UKHPR1000-1276	Methods for non-standard regions	
RQ-UKHPR1000-1277	ANSYS model 2 mesh validation	
RQ-UKHPR1000-1278	Fire barrier rating	
RQ-UKHPR1000-1298	Shielding of and Fire Spread Through Openings	
RQ-UKHPR1000-1299	Worked Examples of Aircraft Impact Perforation Calculations	
RQ-UKHPR1000-1320	IC and IC liner performance under pipe whip loads	
RQ-UKHPR1000-1321	Golden thread for IC liner	
RQ-UKHPR1000-1322	Modelling approach for the IC gusset region	
RQ-UKHPR1000-1323	Flow of safety requirements between schedules	
RQ-UKHPR1000-1324	Undrained shear strength and sliding resistance	
RQ-UKHPR1000-1329	AI vibration damage in BFX	
RQ-UKHPR1000-1330	Consideration of safety system maintenance for AI	
RQ-UKHPR1000-1331	Aircraft impact across multiple buildings	
RQ-UKHPR1000-1332	Vibration qualification of equipment under AI	
RQ-UKHPR1000-1333	Structural acceptance criteria for AI	
RQ-UKHPR1000-1334	Fire spread through underground passages	
RQ-UKHPR1000-1335	Consideration of concrete spalling due to impact on external containment	
RQ-UKHPR1000-1344	Demonstration of SFR Compliance for AI	

RQ Number	RQ Title	Link to GDA Step 3 assessment
RQ-UKHPR1000-1345	Spatial separation of supporting systems	
RQ-UKHPR1000-1357	Credible Solutions for tilt & settlement	
RQ-UKHPR1000-1379	Effect of thickening external wall on fire spread analysis	
RQ-UKHPR1000-1380	Effect of sensitivity studies on design	
RQ-UKHPR1000-1391	Derivation of DLF's for impulsive loads	
RQ-UKHPR1000-1430	Thermal reduction factor for Gusset Region	
RQ-UKHPR1000-1431	Boundary conditions for M1_gusset model	
RQ-UKHPR1000-1432	Damping response level for limit state D buildings	
RQ-UKHPR1000-1433	Shear design for gusset region	
RQ-UKHPR1000-1439	Strain gauge layout for Internal Containment	
RQ-UKHPR1000-1440	Tendon failure detectability study methodology	
RQ-UKHPR1000-1441	Re-anchorage length of tendons	
RQ-UKHPR1000-1477	Soil spring consistency between reports	
RQ-UKHPR1000-1478	Cliff-edge assessment of BFX spent fuel pool and its supporting structure	
RQ-UKHPR1000-1479	Cliff-edge assessment methodology for Internal Containment	
RQ-UKHPR1000-1480	Cliff-edge assessment report clarifications	
RQ-UKHPR1000-1481	Cliff-edge assessment methodology for overall BFX structure	
RQ-UKHPR1000-1482	Equipment hatch modelling interface	
RQ-UKHPR1000-1483	Equipment hatch local failure modes	
RQ-UKHPR1000-1484	ANSYS and ABAQUS model comparison for internal containment	
RQ-UKHPR1000-1485	Equipment hatch local model	
RQ-UKHPR1000-1486	Material properties for ultimate capacity evaluation	
RQ-UKHPR1000-1487	Tendon total strain at ultimate capacity	
RQ-UKHPR1000-1488	Overall methodology for identifying and assessing different IC failure modes for Ultimate Capacity Evaluation	
RQ-UKHPR1000-1489	Ultimate shear capacity calculation for the Internal Containment	
RQ-UKHPR1000-1521	Load combinations for Internal Containment	
RQ-UKHPR1000-1522	Containment fragility curves used for Level 2 PSA	
RQ-UKHPR1000-1523	Consideration of uncertainties for containment fragility assessment	
RQ-UKHPR1000-1524	Containment failure modes considered in Level 2 PSA	
RQ-UKHPR1000-1525	Number of Latin Hypercube samples for containment fragility assessment	
RQ-UKHPR1000-1526	Prestress load effect adjustment for IC	
RQ-UKHPR1000-1533	Design Reference Gap Analysis for Civil Engineering	

RQ Number	RQ Title	Link to GDA Step 3 assessment
RQ-UKHPR1000-1535	Design Substantiation Report for Internal Containment	
RQ-UKHPR1000-1536	Shear calculation of Internal Containment	
RQ-UKHPR1000-1537	Design Substantiation Report for Common Raft Foundation	
RQ-UKHPR1000-1539	Diesel Building hydrocarbon fire design	
RQ-UKHPR1000-1540	Effects of disagreement on fire spread rules	
RQ-UKHPR1000-1565	Forward commitments for Analysis and Design Method Statements	
RQ-UKHPR1000-1567	Queries on revised Fuel Assembly drop analysis	
RQ-UKHPR1000-1568	Seismic code at site-specific phase	
RQ-UKHPR1000-1569	Design Basis Accident loading for Common Raft Foundation	
RQ-UKHPR1000-1570	Methodology for dropped loads assessment	
RQ-UKHPR1000-1585	Consideration of thermal effects in nonlinear structural analysis of Internal Containment	
RQ-UKHPR1000-1586	Generic site envelope bearing capacity	
RQ-UKHPR1000-1587	Modelling approach for hatches in Internal Containment	
RQ-UKHPR1000-1629	Decommisioning method and sequence	
RQ-UKHPR1000-1631	Constructability and conventional safety for the BFX roof (for Civil Engineering)	
RQ-UKHPR1000-1632	Compliance with R3 and ACI 349-13 for local structural design for impactive effects	
RQ-UKHPR1000-1648	Questions on Methodology for Liner Design	
RQ-UKHPR1000-1649	Design Substantiation Report for BNX	
RQ-UKHPR1000-1650	Impact of Fuel Transfer Canal Shielding Detail on Provision for Seismic Joints	
RQ-UKHPR1000-1652	Inconsistencies Between Data Presented in External Hazards and Civil Engineering Safety Case Documents	
RQ-UKHPR1000-1657	Combination effects from consequential seismic / external hazards	
RQ-UKHPR1000-1663	Simulation of material properties in Latin Hypercube Simulation	
RQ-UKHPR1000-1664	Refinements to mesh for ABAQUS global model at site-specific	
RQ-UKHPR1000-1665	Civil Engineering independent peer review	
RQ-UKHPR1000-1666	Boundary conditions for equipment hatch local model	
RQ-UKHPR1000-1667	Outputs from ABAQUS Global Model 2 used for Ultimate Capacity Evaluation	
RQ-UKHPR1000-1706	Reporting of Aircraft Crash Safety Evaluation Results	

Annex 4

Assessment Findings

Number	Assessment Finding	Report Section
AF-UKHPR1000-0214	The licensee shall, as part of detailed design, ensure that the structural analysis models and design information for civil engineering is harmonised consistent with the latest design reference.	4.3.3
AF-UKHPR1000-0215	The licensee shall, as part of the site-specific design, ensure that the civil engineering design requirements include relevant combinations of external hazard and internal hazard loads.	4.4.2
AF-UKHPR1000-0216	The licensee shall, as part of the site-specific design, address the following areas concerning the modelling approach using finite element analysis and the compounding effect on the design: • Validation of the conditioning of the finite element mesh in localised regions. • Demonstration of a systematic methodology for ensuring the design results that are based on simplified centreline models for the shell element of the reinforced concrete structure reliably cater for offsets in the real structure. • Validation to substantiate the omission of joints in the global analysis models and the potential underestimation of stiffness of spanning elements. • For the global analysis models refine the finite element mesh density and the post processing methodology to meet appropriate convergence criteria.	4.4.4
AF-UKHPR1000-0217	The licensee shall, as part of the site-specific design, develop the GDA seismic analysis methodology to fully meet relevant good practice and address the compounding effect on the design of structures, systems and components. This should address the following aspects: • The use of a suite of at least five sets of time histories as per Section 2.6.1 of ASCE4-16 that are selected and modified appropriately to meet the requirements of ASCE4-16 and ASCE43-19. • Validation of the GDA assumptions for the modelling of plant and equipment to justify whether the simplified approach is conservative. • Inclusion of an Importance Factor in accordance with ACI 350.3-06 in the calculation of hydrodynamic loads and freeboard height. • The analysis of dynamic soil pressures on embedded walls within the footprint of a structure. • Capture the out-of-plane response of the floor slabs in the generation of floor response spectra.	4.4.5

Number	Assessment Finding	Report Section
	• Full compliance with ASCE4-16 to ensure the assumed level of structural damping and extent of concrete cracking under seismic loading is appropriate for the structures stress state. • Full articulation of the structure-soil-structure interaction methodology, and analysis and full evaluation of structure-soil-structure interaction effects. • Detailed consideration of embedment effects under site-specific conditions.	
AF-UKHPR1000-0218	The licensee shall, as part of the site-specific design, implement a methodology for demonstrating global stability, incorporating but not limited to the following: • Checks on the drained and undrained shear resistance of the soil and justification for the friction coefficient used to represent any waterproof membrane. • Checks against external explosion loading. • Consideration of concentrated forces on structural protrusions that will act as shear keys. • The criteria for assessing the extent of uplift that is acceptable under seismic loading.	4.4.7
AF-UKHPR1000-0219	The licensee shall, as part of detailed design, resolve the following aspects associated with the strength design methodology: • The methodology for the design of 2D reinforced concrete structures within the post-processing software, is expected to take account of the biaxial stress state when deriving the concrete's strength consistent with relevant good practice. • The methodology for determining the material design strengths that will be used when evaluating complex D-regions using finite element analysis should be fully articulated consistent with relevant good practice.	4.4.7
AF-UKHPR1000-0220	The licensee shall, as part of the site-specific design, resolve the following for the spent fuel pool in the fuel building: • Apply a more refined approach for the determination of hydrodynamic loads in accordance with relevant good practice or provide further justification that the approach adopted is suitably bounding. • Demonstrate that the freeboard allowance is adequate under design basis earthquake conditions in accordance with relevant good practice.	4.5.3
AF-UKHPR1000-0221	The licensee shall, as part of the detailed design of the fuel building, undertake sensitivity studies to demonstrate that the chosen spatial position for the fuel assembly drop load case is bounding in terms of in-structure demands.	4.5.5

Number	Assessment Finding	Report Section
AF-UKHPR1000-0222	The licensee shall, as part of the detailed design of the fuel building, resolve the following for the spent fuel pool liner: • Provide justification for the mixing of design codes for assessing the design resistance of the liner framework system. • Justify the pond concrete durability against any minor undetected leakage over its design life.	4.5.6
AF-UKHPR1000-0223	The licensee shall, as part of the detailed design, reanalyse the structure to demonstrate that the modifications to the fuel building geometry reduce civil engineering risks as low as reasonably practicable.	4.5.7
AF-UKHPR1000-0224	The licensee shall, as part of the site-specific design, demonstrate that the seismic analysis approach for the internal containment captures the local response in a conservative manner. This should include, but not be limited to, the ring belt and dome region of the internal containment.	4.6.4
AF-UKHPR1000-0225	The licensee shall, as part of the site-specific design of the internal containment, validate the application of the thermal reduction factors considering the structures stress state. This should include but not be limited to the common raft foundation under the reactor building and the internal containment standard zone.	4.6.4
AF-UKHPR1000-0226	The licensee shall, as part of the detailed design, justify the overall finite element modelling approach for the internal containment. This should include, but not be limited to, the use of both global and local models, the extent of these models, how they interface and interact, and the validation applied.	4.6.4 & 4.6.5
AF-UKHPR1000-0227	The licensee shall, as part of the detailed design, justify the modelling approach for the internal containment post-tensioned tendons. This should include, but not be limited to, the tendon element constraint.	4.6.5
AF-UKHPR1000-0228	The licensee shall, as part of the detailed design, resolve the following regarding the post-processing of the internal containment analysis results: • Present the actual utilisations for the post-tensioned tendons that explicitly considers the losses along the length of the tendon rather than using an average prestress. • Justify the reduction in tangential shear reinforcement in the standard zone.	4.6.5
AF-UKHPR1000-0229	The licensee shall, as part of the detailed design, refine and validate the strength design methodology for the internal containment gusset to demonstrate that:	4.6.6

Number	Assessment Finding	Report Section
	• the methodology for combining demands on the vertical reinforcement from shear with the axial and bending demands is adequate; and • the methodology for checking concrete stresses within the gusset sub-sections is in accordance with relevant good practice for strut and tie models.	
AF-UKHPR1000-0230	The licensee shall, as part of the detailed design, refine the analysis and design approach for penetrations to demonstrate the internal containment design is adequate under accidental loading arising from high energy pipe failure.	4.6.7
AF-UKHPR1000-0231	The licensee shall, as part of the detailed design, demonstrate that the analysis to derive the internal containment fragility functions is consistent with the deterministic analysis for evaluating the internal containment ultimate capacity and is in accordance with relevant good practice.	4.6.12
AF-UKHPR1000-0232	The licensee shall, as part of the site-specific design, optimise the design of the common raft foundation to satisfy the global stability requirements.	4.7.4
AF-UKHPR1000-0233	The licensee shall, as part of the site-specific design, substantiate the design of the common raft foundation to ascertain the impact of the detailed geometrical configuration on the areas of the raft with high utilisation. This should include, but not be limited to, the pits adjacent to the internal containment gusset area.	4.7.5
AF-UKHPR1000-0234	The licensee shall, as part of the detailed design, justify the methodology for the substantiation of the claimed barriers against hydrocarbon fires.	4.8.5
AF-UKHPR1000-0235	The licensee shall, as part of the detailed design for malicious aircraft impact, demonstrate that the protective structures over the seismic gaps prevent fuel from penetrating between the buildings. This should include, but not be limited to, the gap between Safeguard Building C and the External Containment.	4.10.5

Annex 5

A. Description of civil engineering structures within GDA Step 4 Scope

A.1 UK HPR1000 GDA configuration:

A.1.1 The generic site layout for the UK HPR1000 is based on a single unit reactor design. The structures that are within the scope of GDA include, illustrated in Figure A.1.1 below:

- Reactor Building (BRX), which includes BRX internal structures, internal containment and external containment
- Safeguard buildings, which include Safeguard Building A (BSA), Safeguard Building B (BSB), Safeguard Building C (BSC)
- Fuel Building (BFX)
- Nuclear Auxiliary Building (BNX)
- Diesel generator buildings, which include Emergency Diesel Generator Building A (BDA), Emergency Diesel Generator Building B (BDB), Emergency Diesel Generator Building C (BDC), SBO Diesel Generator Building for Train A (BDU) and SBO Diesel Generator Building for Train B (BDV)
- Radioactive Waste Treatment Building (BWX)
- Extra Cooling System and Fire-fighting Water Production System Building (BEJ)
- Equipment Access Building (BEX)
- Personnel Access Building (BPX)

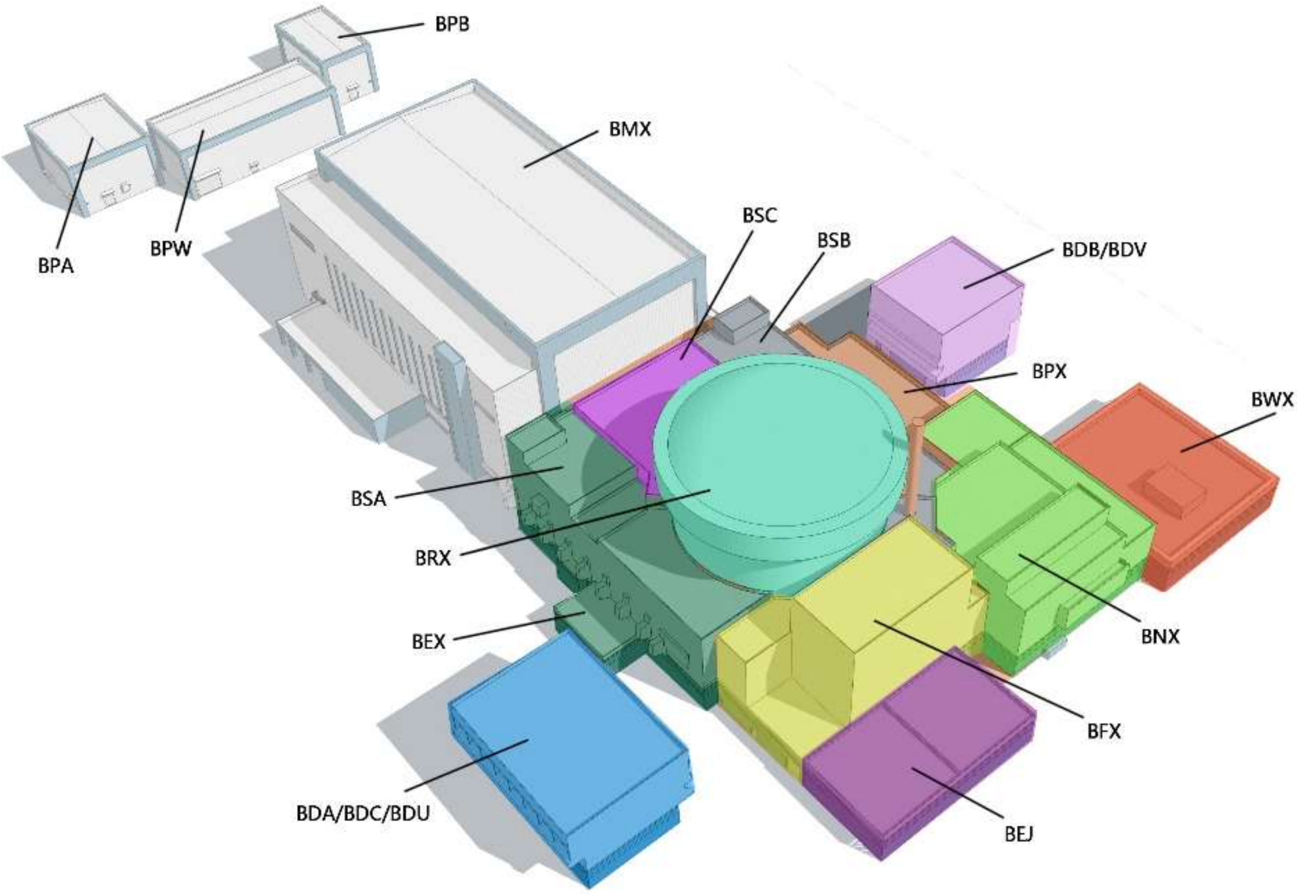


Figure A.1.1: An illustration of the generic UK HPR1000 design - the structures illustrated in colour are within GDA scope

A.1.2 The structures that are outside the scope of GDA, (coloured grey in Figure A.1.1 above) include:

- Essential Service Water Pumping Station A (BPA) and Essential Service Water Pumping Station B (BPB)
- Circulating Water Pumping Station (BPW)
- Essential Service Water Supply Gallery A (BGA), Essential Service Water Supply Gallery B (BGB), Essential Service Water Supply Gallery C (BGC)
- Diesel Buildings Integrated Gallery H (BGH), Diesel Buildings Integrated Gallery I (BGI), Diesel Buildings Integrated Gallery J (BGJ)
- Essential Service Water Drain Gallery L (BGL), Essential Service Water Drain Gallery M (BGM), Essential Service Water Drain Gallery N (BGN)
- Turbine Generator Building (BMX)

A.1.3 The buildings located on the common raft foundation that forms the Nuclear Island include the BRX, BFX and Safeguard Buildings, whereas the BNX, BWX, BEJ, BPX, BEX, BDA/BDC/BDU and BDB/BDV are located on independent raft foundations. The structural relationship of buildings on common raft foundation is described as follows:

- The External Containment, BFX and Safeguard Buildings are connected together below the 0.00 m elevation;
- The BSA-II, External Containment and BSB-II are connected together above the 0.00 m elevation;
- The common raft building is separated from the adjacent buildings by a 200 mm gap (see the dashed red and green lines in Figure A.1.2 below). These are denoted *'Type 2'* joints for the seismic joint calculations.
- The BFX, BSA-II + External Containment + BSB-II and BSA-I + BSC + BSB-I are separated from each other by 100 mm structural joints above ±0.00 m elevation (see the solid red lines in Figures A.1.2 and A.1.3 below). The BRX internal structure and internal containment are separated from each other by 200 mm structural joints above the -2.600m elevation (see the solid green line in the Figures A.1.2 and A.1.3 below). These are denoted *'Type 1'* joints for the seismic joint calculations.

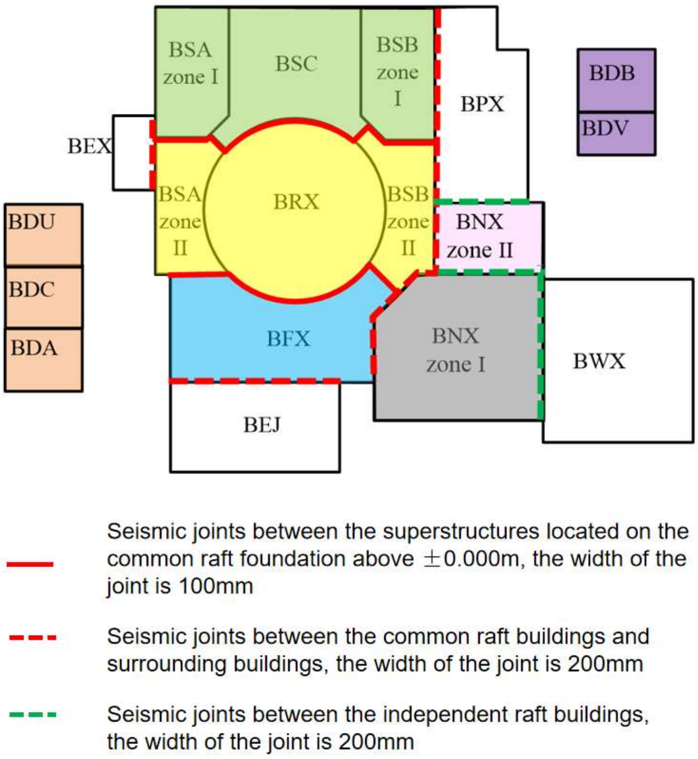


Figure A.1.2: Plan view with seismic joints illustrated to identify locations of joints and joint type

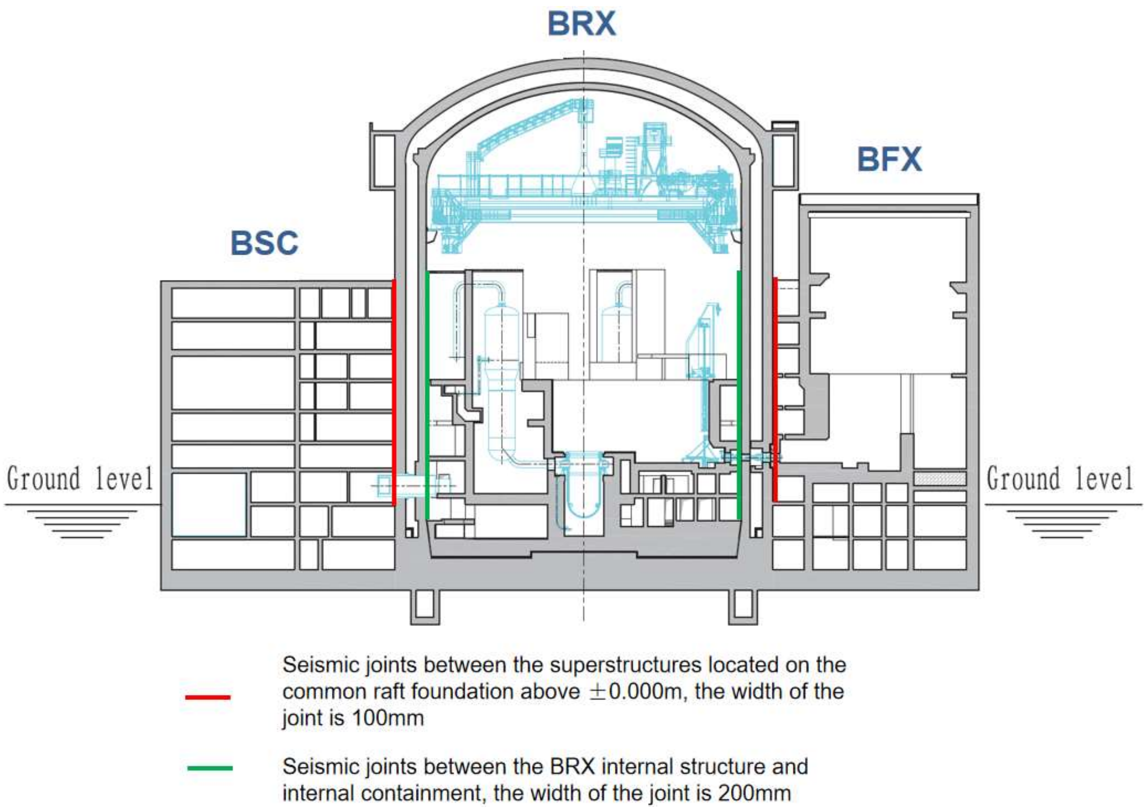


Figure A.1.3: Elevation view with seismic joints illustrated to identify locations of joints and joint type

<u>Description of Sample Structures</u>

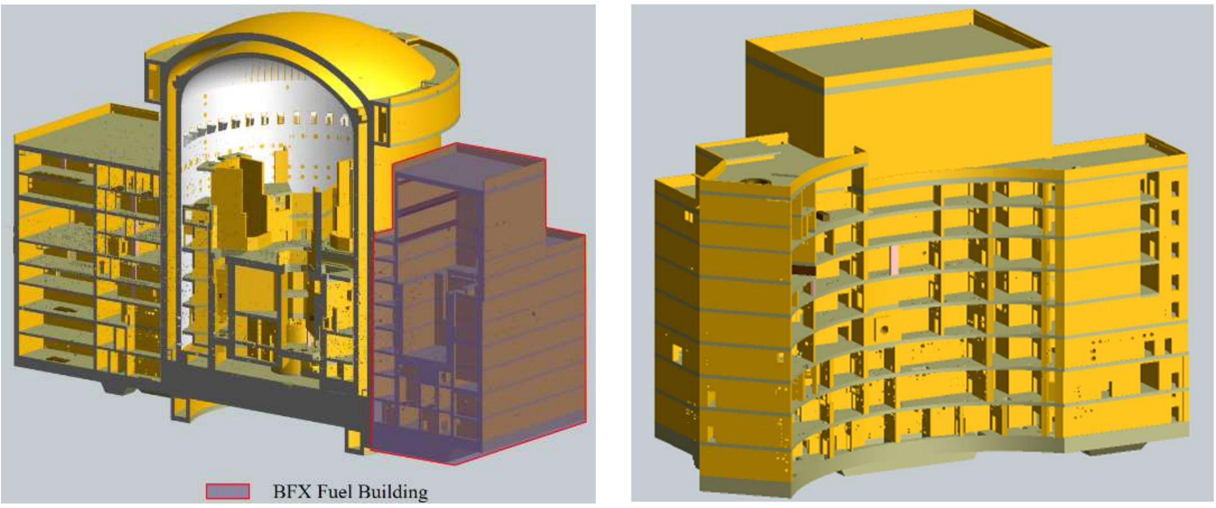
A.1.4 For the purposes of this assessment report, the sample structures are described in more detail in the following sections to provide useful background and context for the reader. These structures are the BFX, Internal Containment, and Common Raft. The Aircraft Impact protection is not described here for security reasons.

A.2 BFX

<u>Overview</u>

A.2.1 The BFX share the common raft foundation with the BRX and Safeguard Buildings. The BFX is adjacent to the external containment, BSA, BSB, BEJ and BNX. The main function of BFX is to provide accommodation for the storage, transfer and lifting of new nuclear fuel and spent fuel.

A.2.2 The BFX is illustrated by the following 3D images in Figure A.2.1 below.



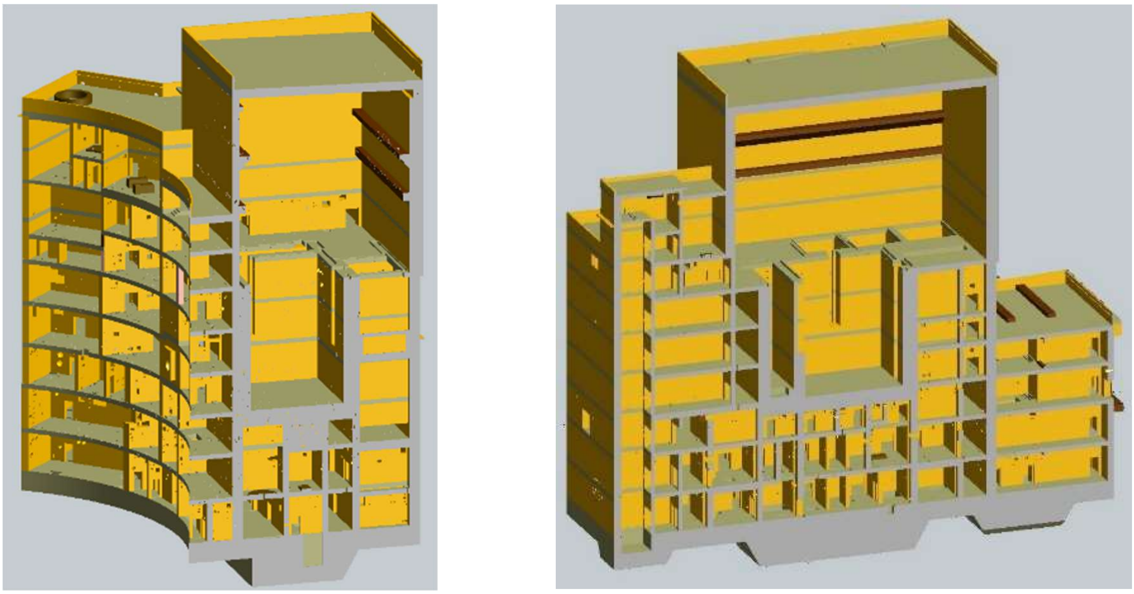


Figure A.2.1: 3D view of the BFX building, illustrating the cellular construction and the interface with BRX

Function

A.2.3 The main function of BFX is to provide accommodation for the storage, transfer and lifting of new nuclear fuel and spent fuel. The Spent Fuel Pool (SFP) located at the middle of the BFX provides cooling to the spent fuel assemblies.

A.2.4 The BFX is divided into three principal functional areas as listed below, see Figure A.2.2 below:

- The mechanical equipment zone.
- The fuel handling and storage zone.
- The ventilation and air conditioning equipment zone.

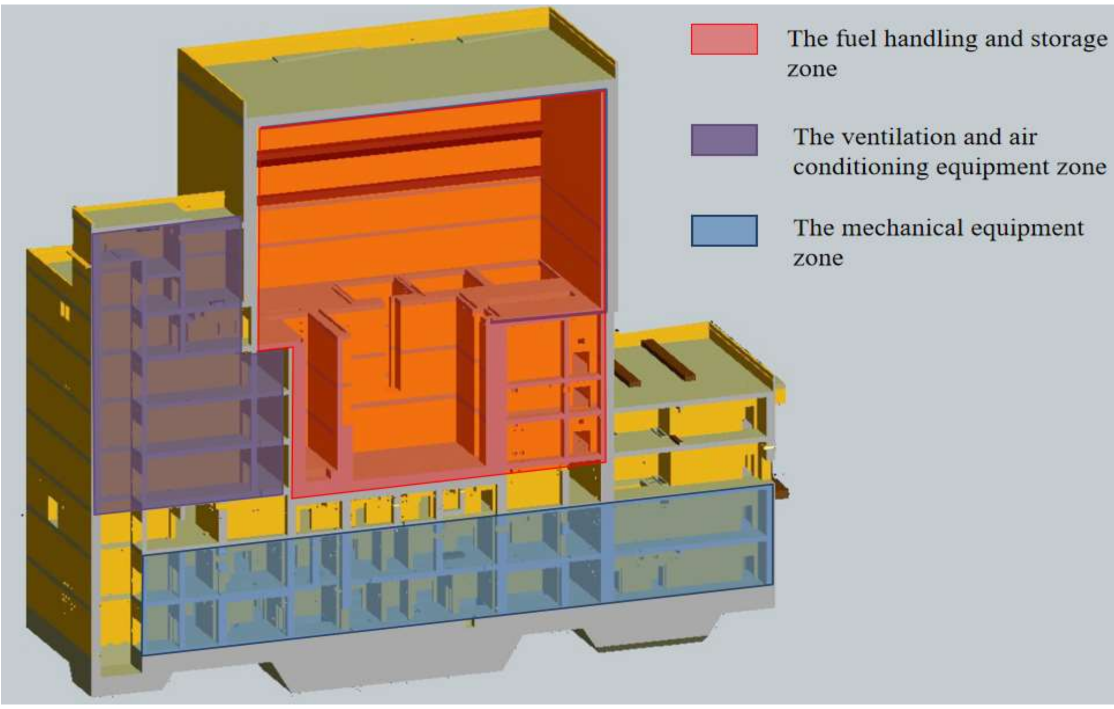
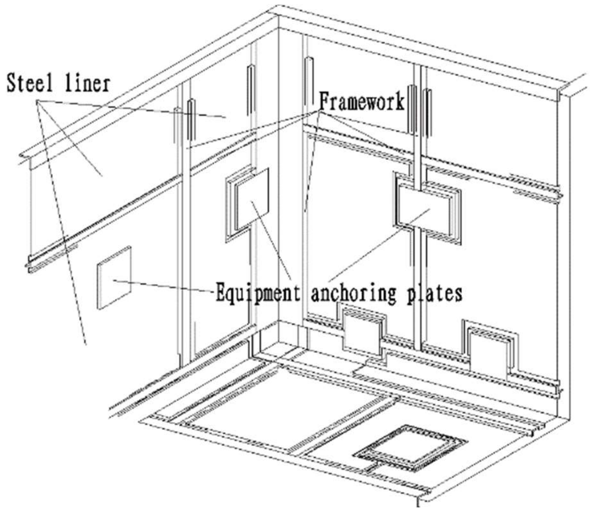


Figure A.2.2: 3D illustration of the BFX building, cutting through the elevation to indicate functional 'zones'

Structural Description

A.2.5 The BFX is a reinforced concrete structure with walls and slabs. The external and internal walls are important vertical force transfer components. The walls are located as evenly as possible and maintain the continuity along vertical direction because this provides a clear load path from the roof to the raft foundation. The BFX is structurally connected with the BSA, BSB and the BRX at the elevation of ±0.000m and below.

A.2.6 The reinforced concrete structure of the spent fuel pool (see Figure A.2.3 below) has a stainless liner to ensure leak-tightness of the pool. This liner is 4mm to 6mm thick and specified as EN 1.4307 austenitic stainless steel. The liner is attached to a framework system and the leakage detection system is deployed behind the liner.



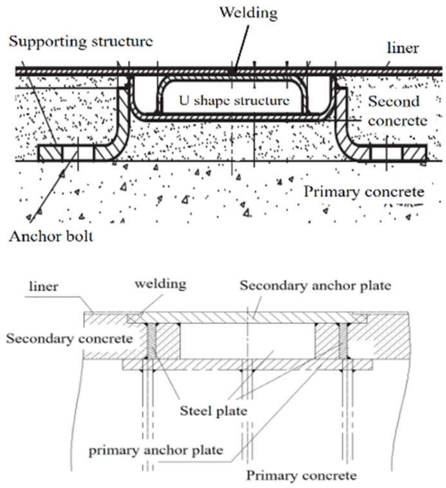


Figure A.2.3: Diagrams of spent fuel pool liner, indicating liner connections to the primary concrete with anchors

Load Paths

A.2.7 Loads of BFX are transferred by roofs and slabs to walls, and then to the common raft foundation.

A.2.8 The vertical loading on the roof and slabs are transmitted to the common raft foundation through the external and internal concrete shear walls.

A.2.9 The horizontal loads which act on the external walls are transmitted to the common raft foundation through the external and internal concrete shear walls and diaphragm slabs. The horizontal and vertical load path are shown in Figure A.2.4 below:

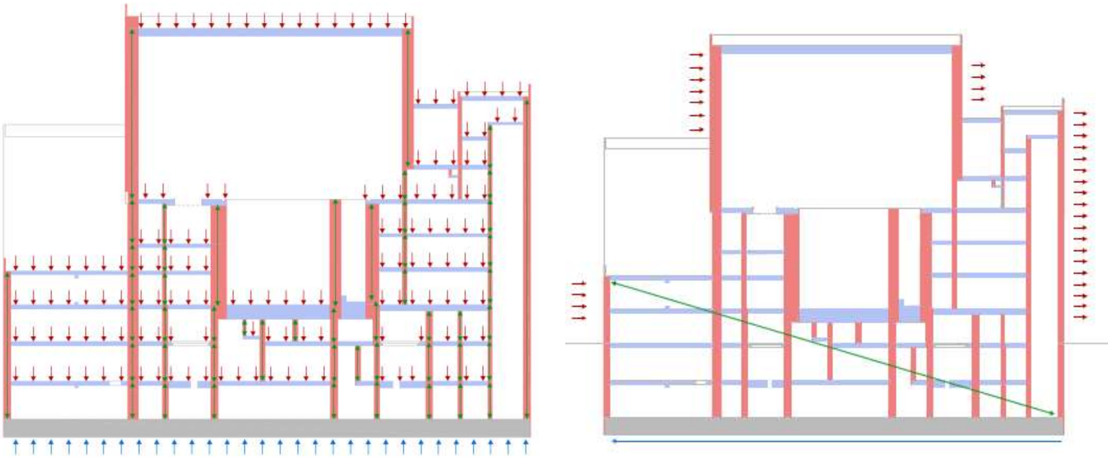


Figure A.2.4: Elevation of the BFX showing the load applications for analysis

Design modification

A.2.10 The proposed design modification of the BFX has been driven by changes to the fuel route and also the need to provide increased access for EIMT activities that comply with UK conventional Health and Safety expectations; see RO-UKHPR1000-0014 and RO-UKHPR1000-0056 (Ref. 7). This required the BFX to increase in size, as graphically shown in Figure A.2.5 below.

A.2.11 The major changes are summarised as:

- The western wall of the fuel handling hall is moved further to the west and becomes coaxial with the western external wall beginning at the foundation level;
- The southern external wall is moved further to the south;
- The elevation of the roof of the fuel handling hall is increased;
- The internal walls are moved further to the south.

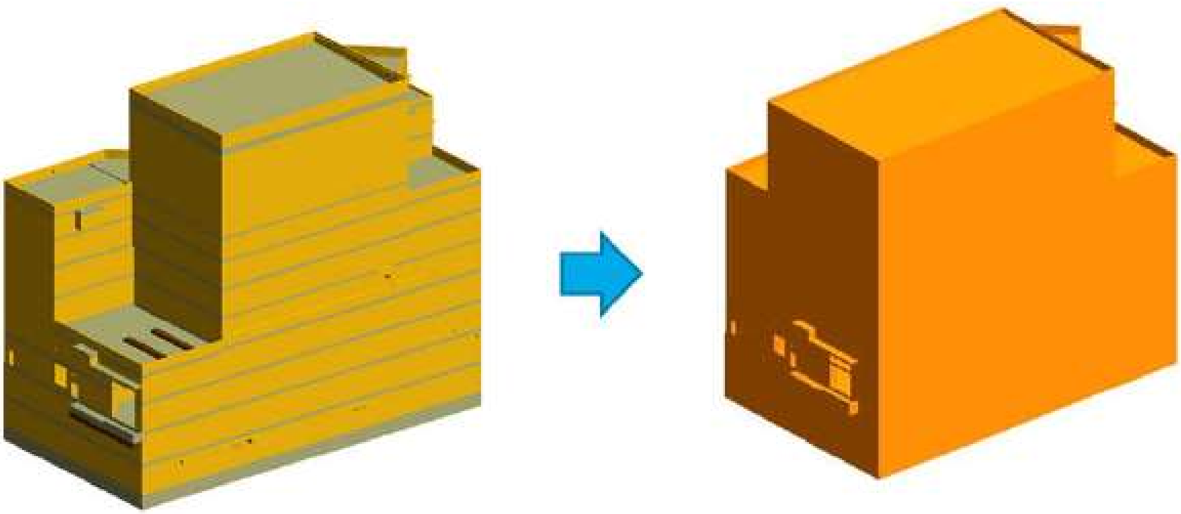


Figure A.2.5: 3D illustration of BFX before modification (left) and after modification (right)

A.2.12 The comparison of sections of the BFX, before after modification are shown in Figures A.2.6 and A.2.7 below.

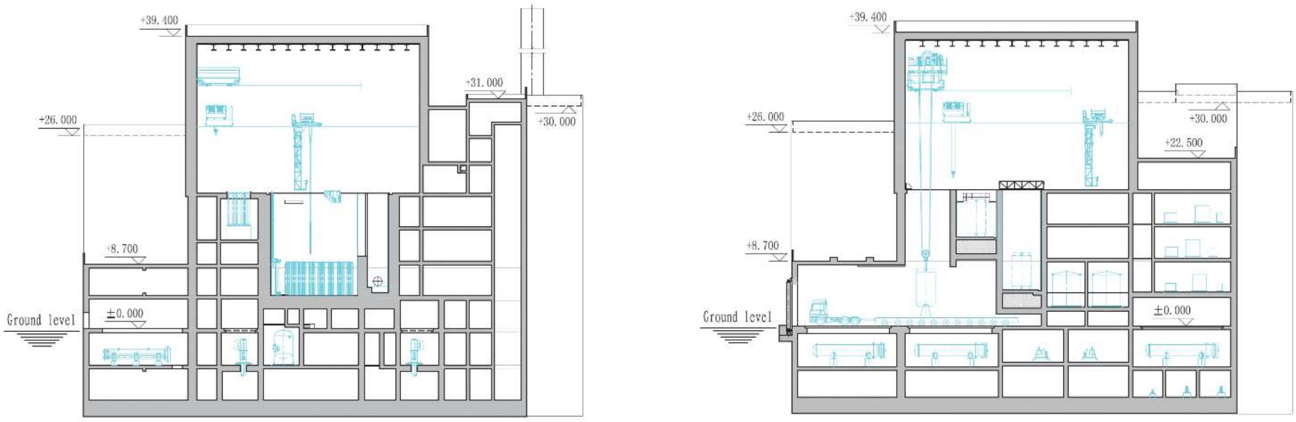


Figure A.2.6: Elevation views of BFX building before modification

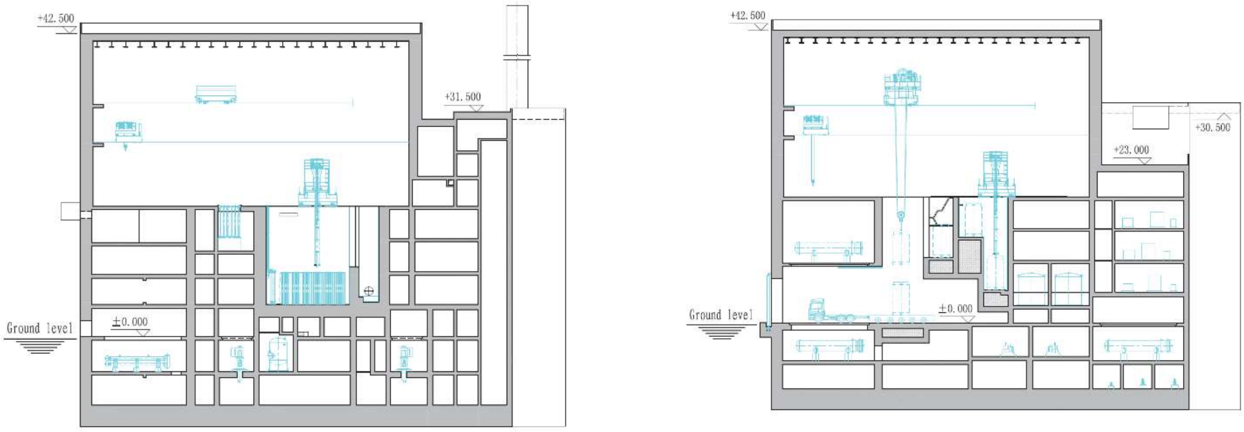


Figure A.2.7: Elevation view of BFX building after modification

A.3 Internal Containment

<u>Overview</u>

A.3.1 The internal containment is part of the BRX facility. The boundary of the BRX comprises the internal containment and external containment structures. These are located on, and in the centre, of the common raft foundation. The external containment is located outside of the internal containment and separated by an annulus of width ≈ 1.8m.

A.3.2 The BRX internal structures are located within the internal containment. There main functions of these structures are to support the reactor pressure vessel and equipment; provide radiological shielding to personnel and protection to equipment; and to provide anti-whip and missile protection for the internal containment, primary and secondary circuits and protection systems.

A.3.3 The external containment is located outside of internal containment. It is a reinforced concrete structure composed of a cylindrical wall and dome. A water tank for the Secondary Passive Heat Removal System is located on the top of the cylindrical walls. The external containment provides protection against external hazards, such as aircraft impact.

A.3.4 A pre-stressing gallery is located below common raft foundation for the tensioning of the pre-stressing tendons. This gallery is structurally connected and continuous with the common raft foundation.

A.3.5 The internal and external containments are illustrated by the following 3D images, Figure A.3.1 below.

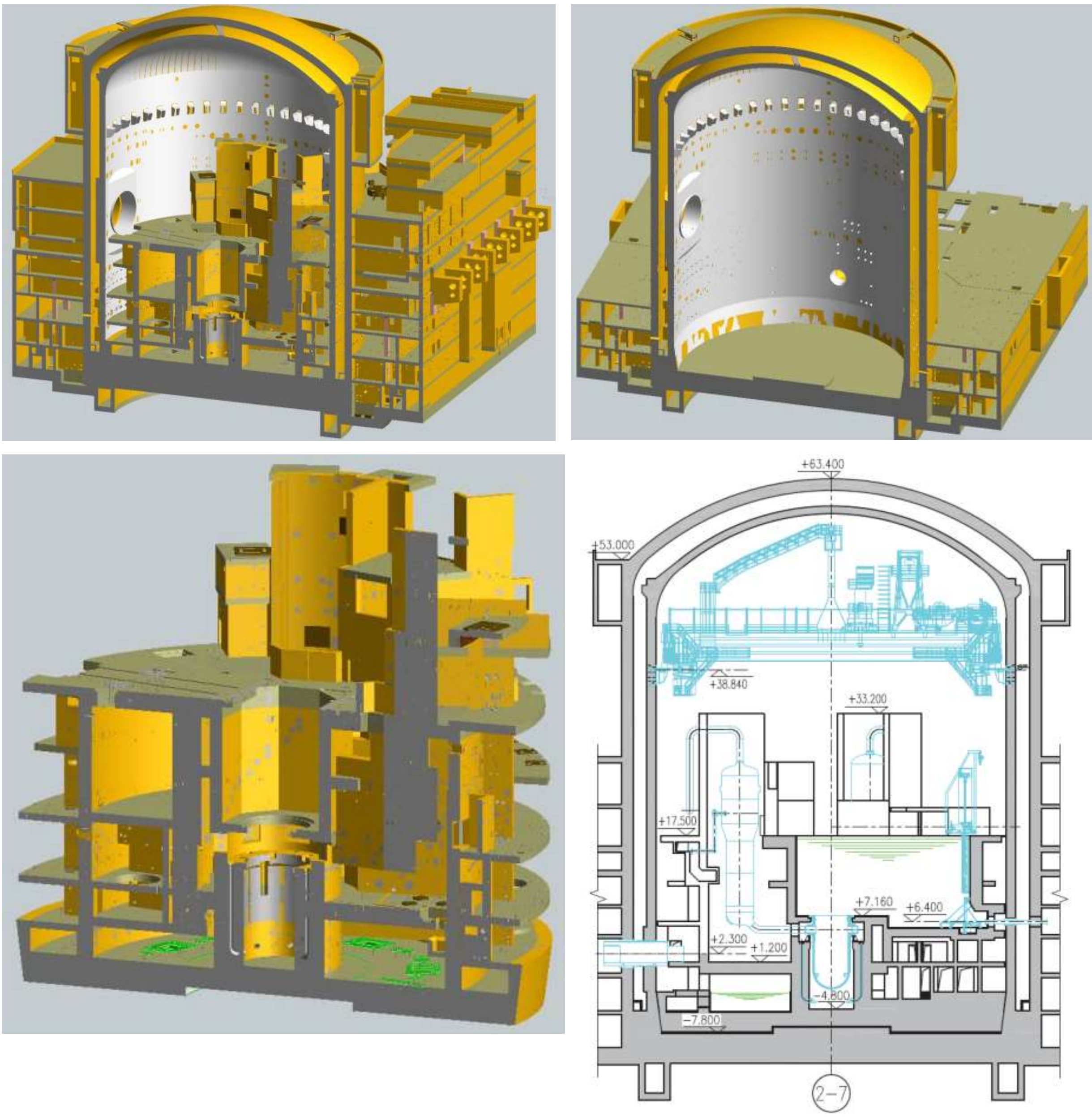


Figure A.3.1: 3D views of the BRX building, and an elevational drawing view of the section through

Function

A.3.6 The Internal containment provides protection against internal and external hazards as well as a shielding function. However, the main function is to provide containment of gases under design basis pressure and temperature conditions, thereby preventing release of radiological material. Furthermore, the internal containment is expected to have substantial beyond design basis capacity to control the release of radiological material.

Structural Description

A.3.7 The internal containment is a pre-stressed concrete structure, with a steel liner covering the internal surface and providing the primary containment function. To facilitate the transfer of equipment, personnel and pipes, an equipment hatch, two personnel access airlocks and other small penetrations are provided through the internal and external containment. The Polar Crane support brackets are located on the internal containment.

A.3.8 The geometry of the internal containment is:

- The internal diameter is approximately 45m;
- The height is approximately 68m;

- The thickness of cylindrical wall and dome respectively are about 1.2m and 1.0m;

A.3.9 The internal containment consists of a cylindrical wall, dome, ring belt, ribs, pre-stressing gallery, and enhanced parts around hatches and penetrations. The ring belt connects the cylindrical wall and dome. Two vertical ribs are located on the cylindrical wall set symmetrically apart. The ring belt, pre-stressing gallery and ribs are designed for the anchorage and installation of pre-stressing tendons. The thickened areas around the hatches and penetrations are due to the more highly concentrated stresses and deviations of the tendons.

A.3.10 Screenshots of the 3D model illustrating the internal containment are shown in Figure A.3.2 below.

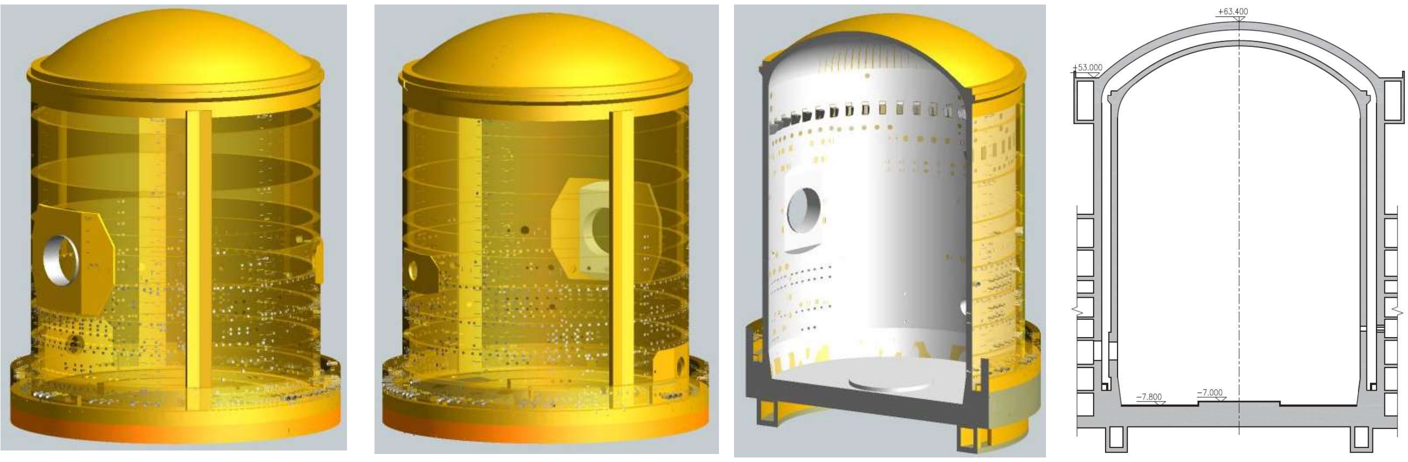


Figure A.3.2: 3D view of the liner for the reactor building containment, with the liner metal illustrated in grey and the reinforced concrete in orange, with an elevation drawing view of the section

A.3.11 The internal containment pre-stressing system includes horizontal and vertical tendons. The horizontal tendons run around the vertical walls. These are anchored to the ribs and tensioned at the two ends. The vertical tendons include "r" tendons and perpendicular tendons. The perpendicular tendons are anchored within the pre-stressing gallery and run vertically up to the ring belt; they are tensioned at one end. The "r" tendons anchor within the pre-stressing gallery and run up the walls and across the dome to anchor at the opposite side at the ring belt; they are tensioned from the two ends. Following tensioning the tendons will be fully protected by installation of a cementitious grout. The behaviour of the inner containment throughout its design life is monitored by a system of strain gauges that is embedded during construction.

A.3.12 The tendon layout is shown in the illustrations in Figure A.3.3 below.

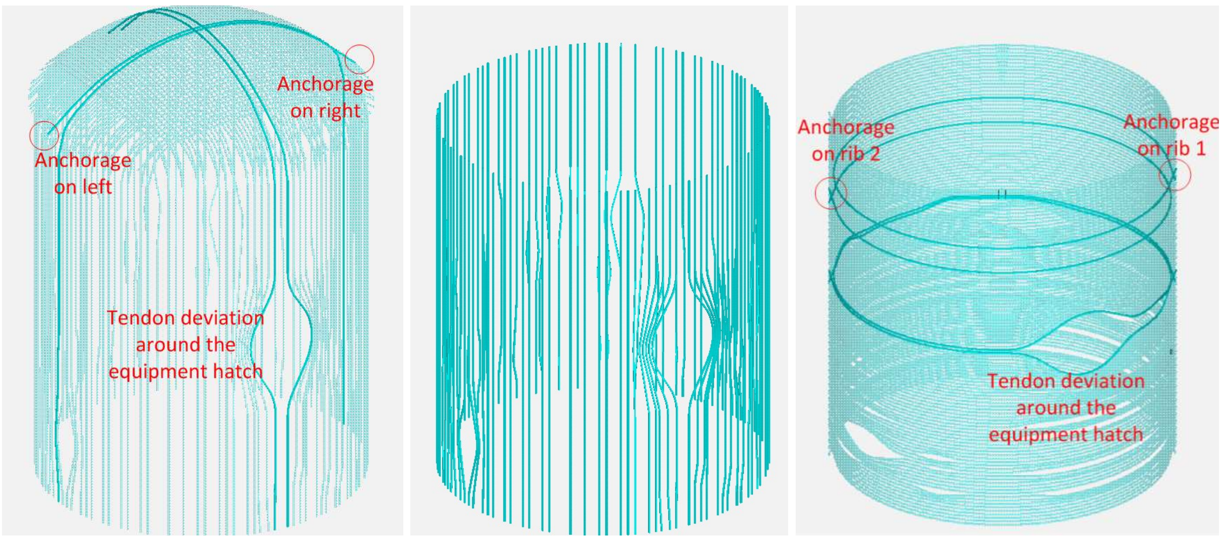


Figure A.3.3: 3D view of the tendons, with the tendon anchorages and deviations annotated

A.3.13 The steel liner is anchored into the internal concrete surface of the internal containment and the top of the Reactor Building foundation. The internal containment liner ensures the leak tightness of the containment is achieved. Furthermore, during construction it forms the inner formwork for constructing the internal concrete containment. The liner anchoring system comprising stiffeners and shear studs are welded to the steel liner plate. It is composed of grids at right angles. The anchoring system is used to reinforce the steel liner and provide stability during the construction and operation periods. It also serves as the construction formwork for the internal containment wall.

A.3.14 Screenshots of the 3D model illustrating the liner are shown in Figure A.3.4 below, with a drawing showing the stud detail for the liner plate in Figure A.3.5 below.

Figure A.3.4: 3D view of the liner for the reactor building containment, with the liner metal illustrated in grey and the reinforced concrete in orange

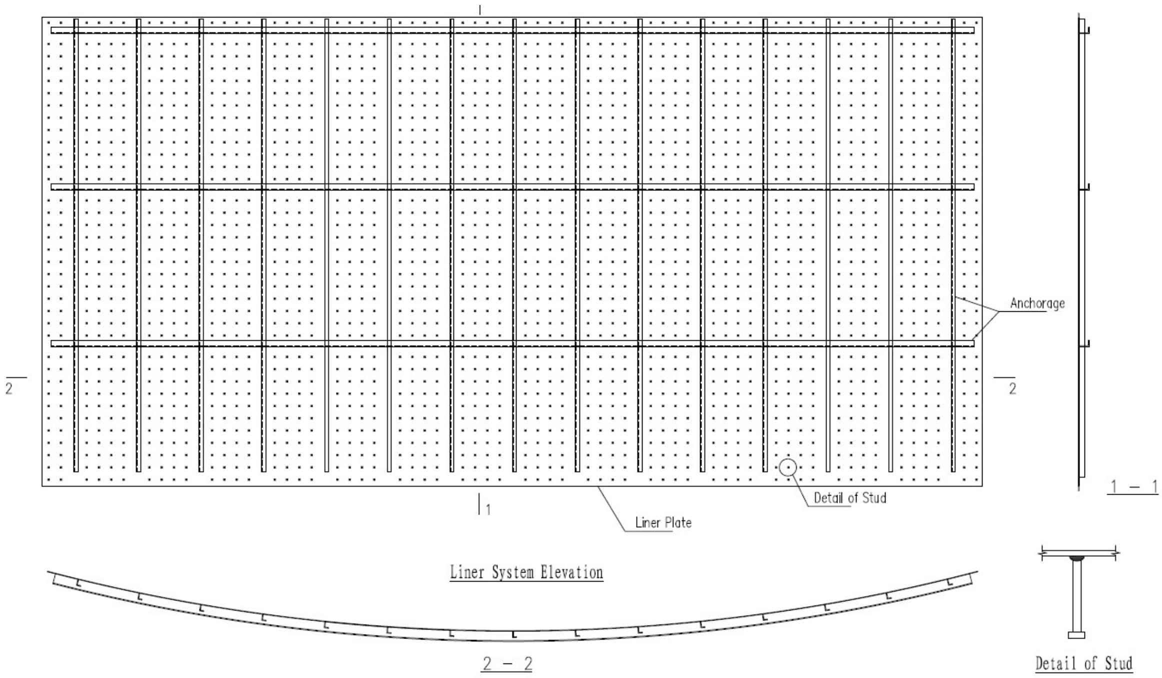


Figure A.3.5: Drawing showing the stud detail for the liner plate

A.3.15 The lower part of the internal and external containment cylindrical walls is enhanced as a gusset which connects the containments and raft foundation. The function of the gusset is to resist the lateral force from internal structures induced by seismic and thermal loading. A raised circular lug or boss is set into the top of the Reactor Building foundation to provide resistance against the lateral forces (induced from seismic motions) from internal structures. The gusset region is illustrated by Figures A.3.6 and A.3.7 below.

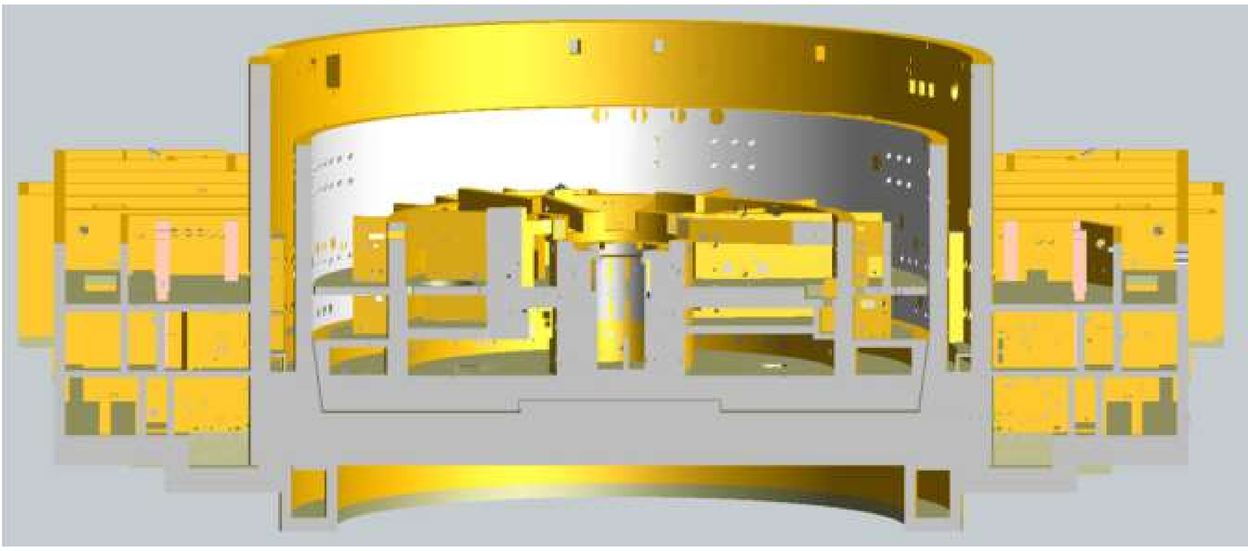


Figure A.3.6: 3D cut-through image of the reactor building, internal and external containment structures alongside the adjacent buildings, with raft foundation profile shown.

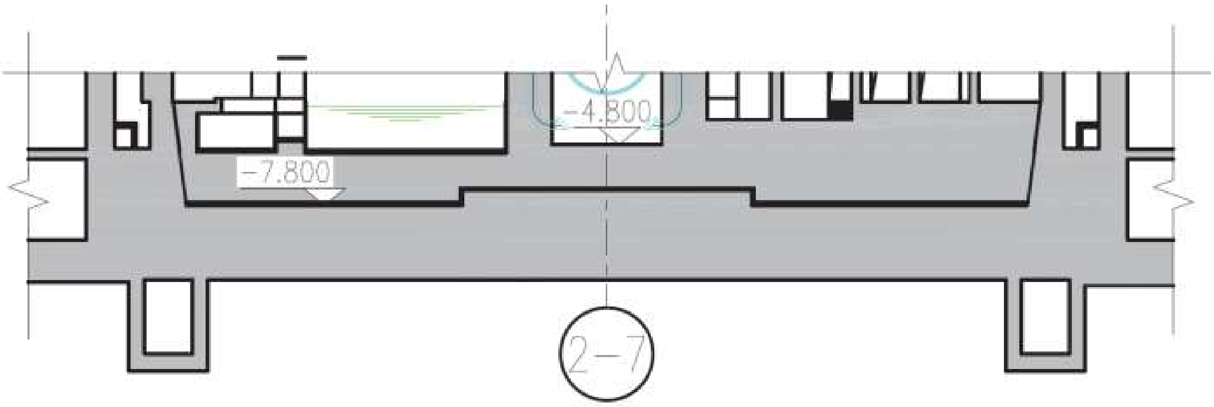


Figure A.3.7: Extract of a drawing of the elevation view, showing the raft foundation profile and prestressing gallery

<u>Load Paths</u>

A.3.16 The load path of the containment is relatively simple because of the geometry feature. The containment is simply a cylinder wall with dome cover, and the cylinder part is not directly connected to the internal structure, so the vertical or lateral loading is transferred downward to the gusset and to the raft foundation. The gusset region is in contact with the internal structure slab, the action resisted by the gusset is mainly the contact pressure due to seismic and thermal loading, which is transferred to the foundation but also partially to the surrounding buildings.

A.3.17 The load path of the containment is illustrated in Figure A.3.8 below.

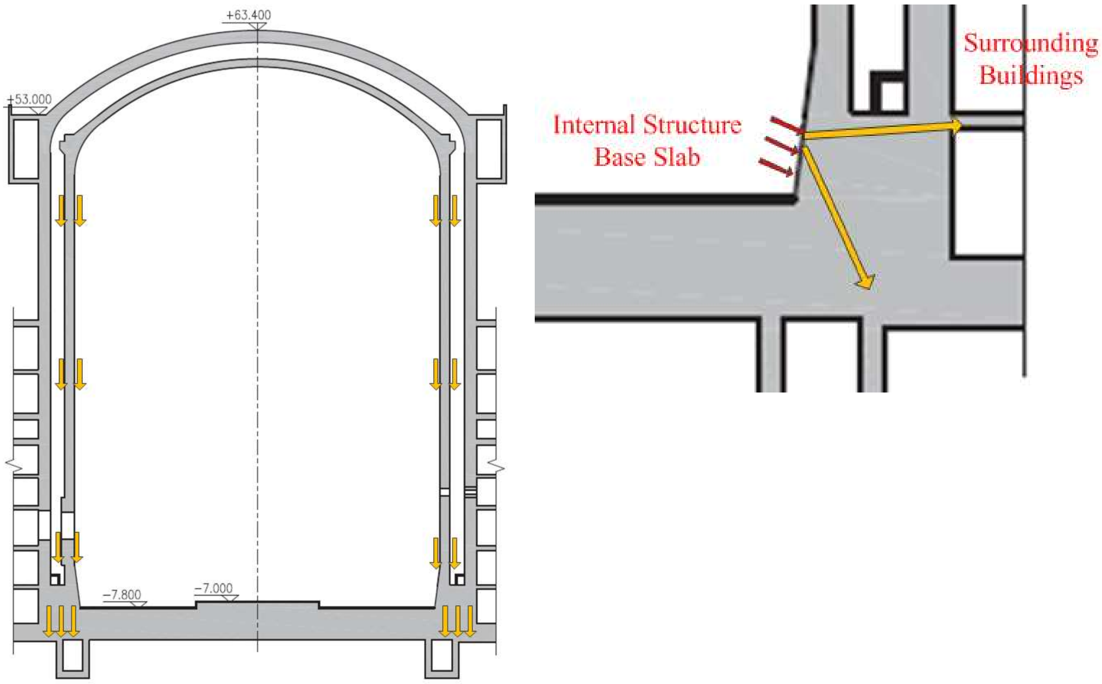


Figure A.3.8: Elevation view showing the load path of the containment, with load path at wall/foundation

A.4 Common Raft Foundation

<u>Overview</u>

A.4.1 To achieve better stability and prevent differential settlement between the buildings, the Reactor Building (BRX), Fuel Building (BFX) and Safeguard Buildings A (BSA), Safeguard Buildings B (BSB) and Safeguard Buildings C (BSC) are built on a common raft foundation. The common raft foundation is a reinforced concrete slab structural element that interfaces between the building structures and the soils. This structure forms a barrier to the potential release of radiological material to the ground and is fundamental to the integrity of the structures and systems it supports.

A.4.2 There are some pits located in the local parts of the common raft foundation, particularly below the Safeguard Buildings and the Fuel Building.

A.4.3 A prestressing gallery is located below common raft foundation for the tensioning of the pre-stressing tendons. This gallery is structurally connected with the common raft foundation.

A.4.4 The common raft foundation is illustrated by the following plan and 3D images shown in Figures A.4.1 and A.4.2 below.

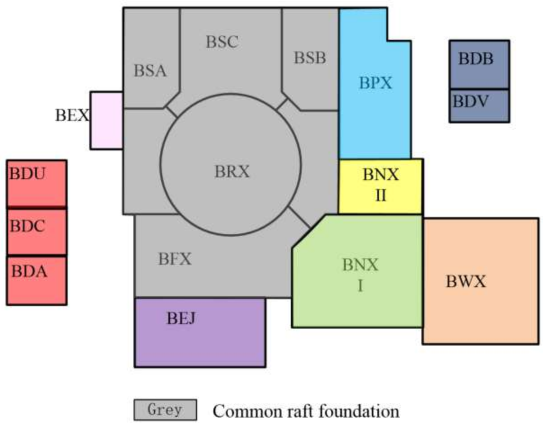


Figure A.4.1: Plan view of GDA scope structures, those coloured grey are supported by the common raft foundation

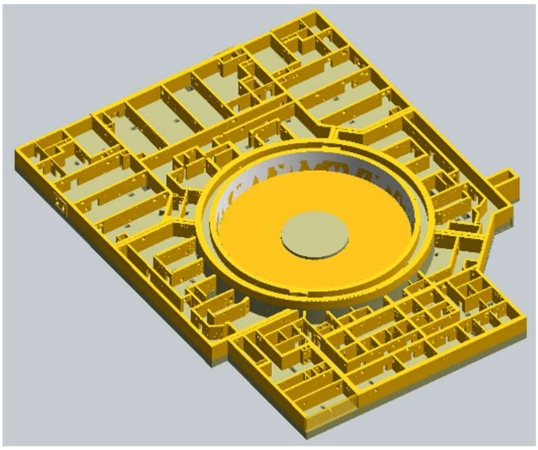


Figure A.4.2: 3D illustration of the common raft foundation and the associated structures it supports

<u>Function</u>

A.4.5 As a structural element, the common raft provides protection against internal and external hazards, as well as keeps the buildings stable, and transfers the loads from the superstructures to the soils. The common raft foundation serves as a final barrier for environmental protection, by preventing soil contamination in case of structure failure, performing two barrier functions:

- protecting the groundwater from any risk of contamination,
- protecting the structures from the groundwater in case of external flooding.

A.4.6 Furthermore, the common raft under BRX is expected to have substantial beyond design basis capacity to control the release of radiological material.

<u>Structural Description</u>

A.4.7 The size of the common raft foundation is about 110 m (length) × 82 m (width). The thickness of the raft varies as follows:

- Under the BRX building, the common raft foundation is 4.65 m thick and there is a 0.8 m high convex set in common raft foundation with a radius of 7.5m, which is designed to prevent sliding from the seismic action.
- Under the Safeguard Buildings and Fuel Building, the common raft foundation is 3.00 m thick.

A.4.8 The underside of the common raft is situated at -12.60 m, see Figures A.4.3 and A.4.4 below:

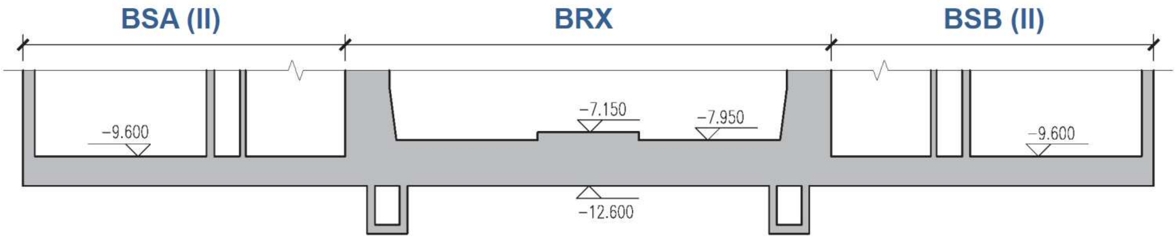


Figure A.4.3: Elevation view of the common raft foundation profile underneath BRX, BSA and BSB

Load Paths

A.4.9 The main force transfers components from the superstructures to the common raft foundation, and vice versa, are the vertical structural elements of the various supported buildings.

A.4.10 The combination of loads from the structures above and the site conditions subject the common raft foundation to bending and shear effects.

A.4.11 Furthermore, there are concentrated prestressing loads anchored on the underside of the common raft foundation which forms the ceiling of the prestressing gallery, shown in green arrows in Figure A.4.4 below.

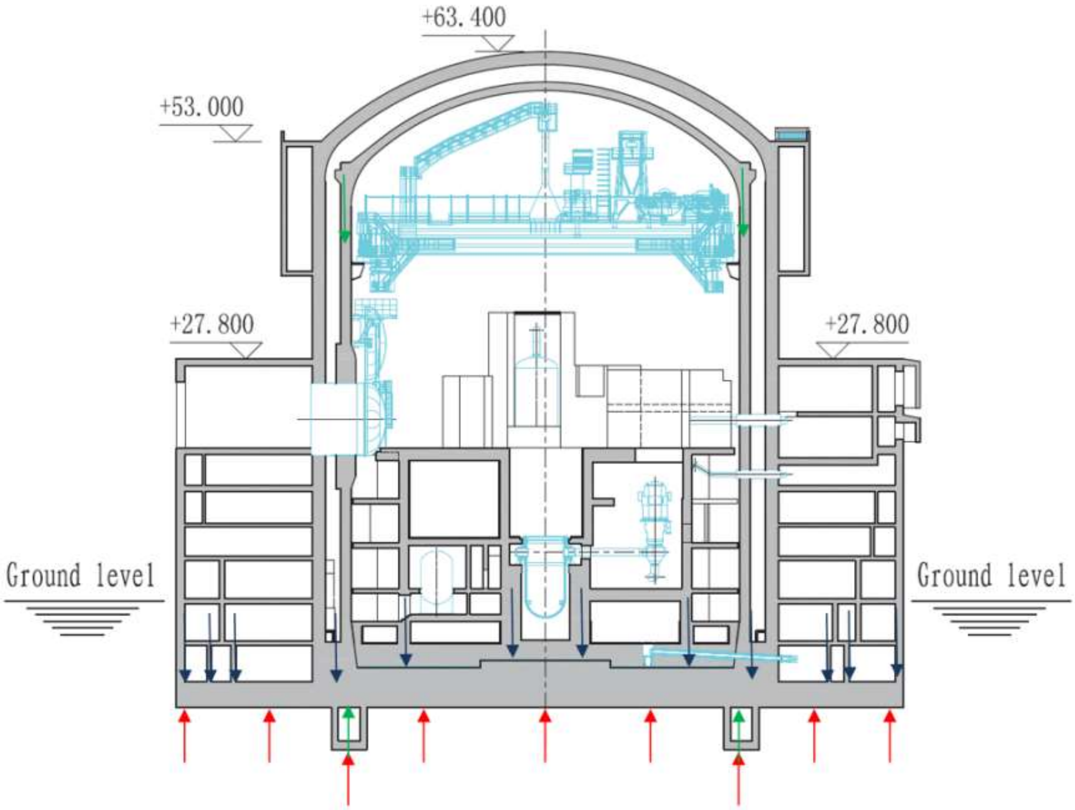


Figure A.4.4: Elevation view of the BRX with adjoining structures, illustrating load paths through common raft foundation

Annex 6

Figures used within the main document text, with large (accessible) print

Figure 1: Golden thread of civil engineering safety cases

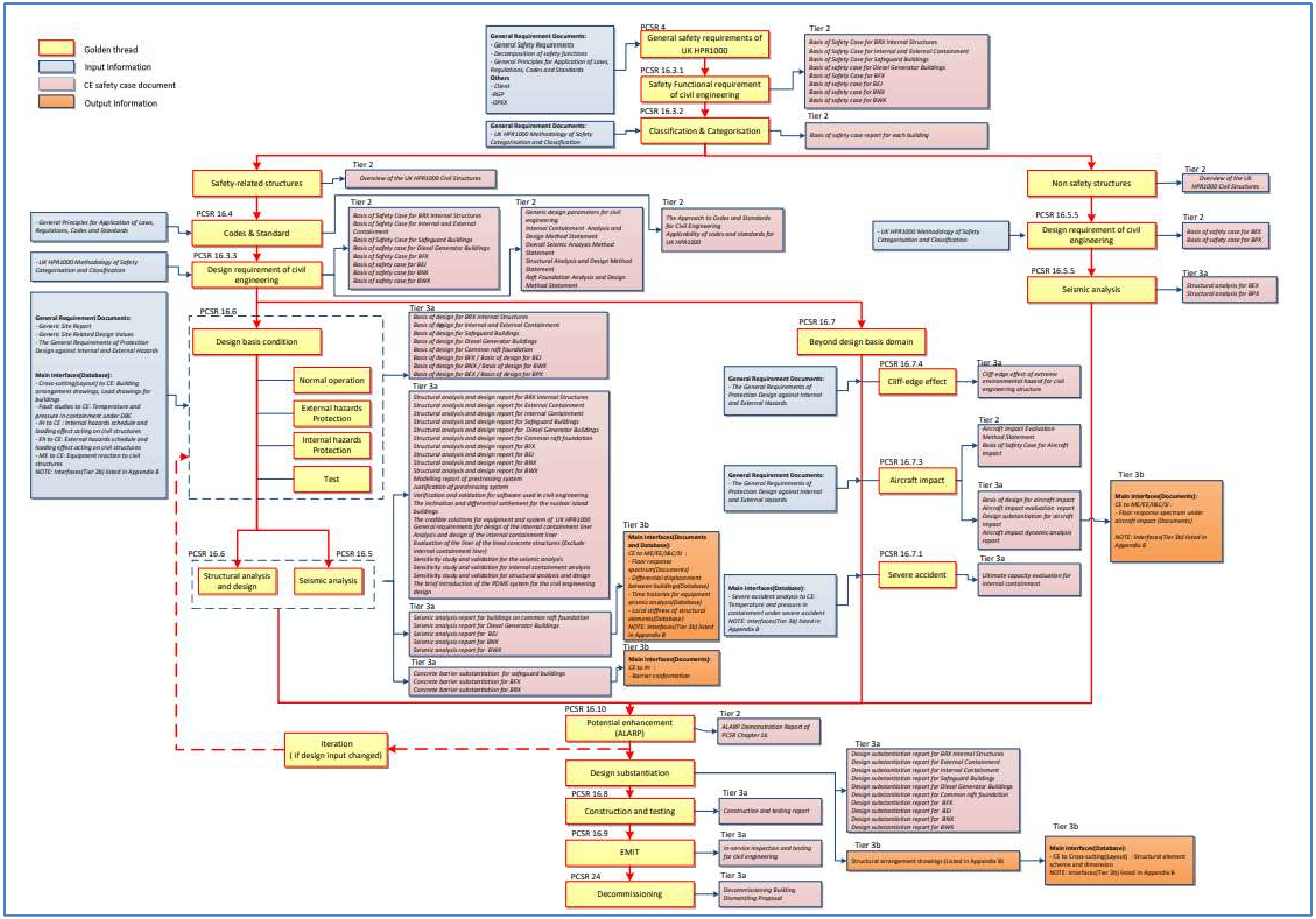


Figure 2: Documentation Map of Civil Engineering

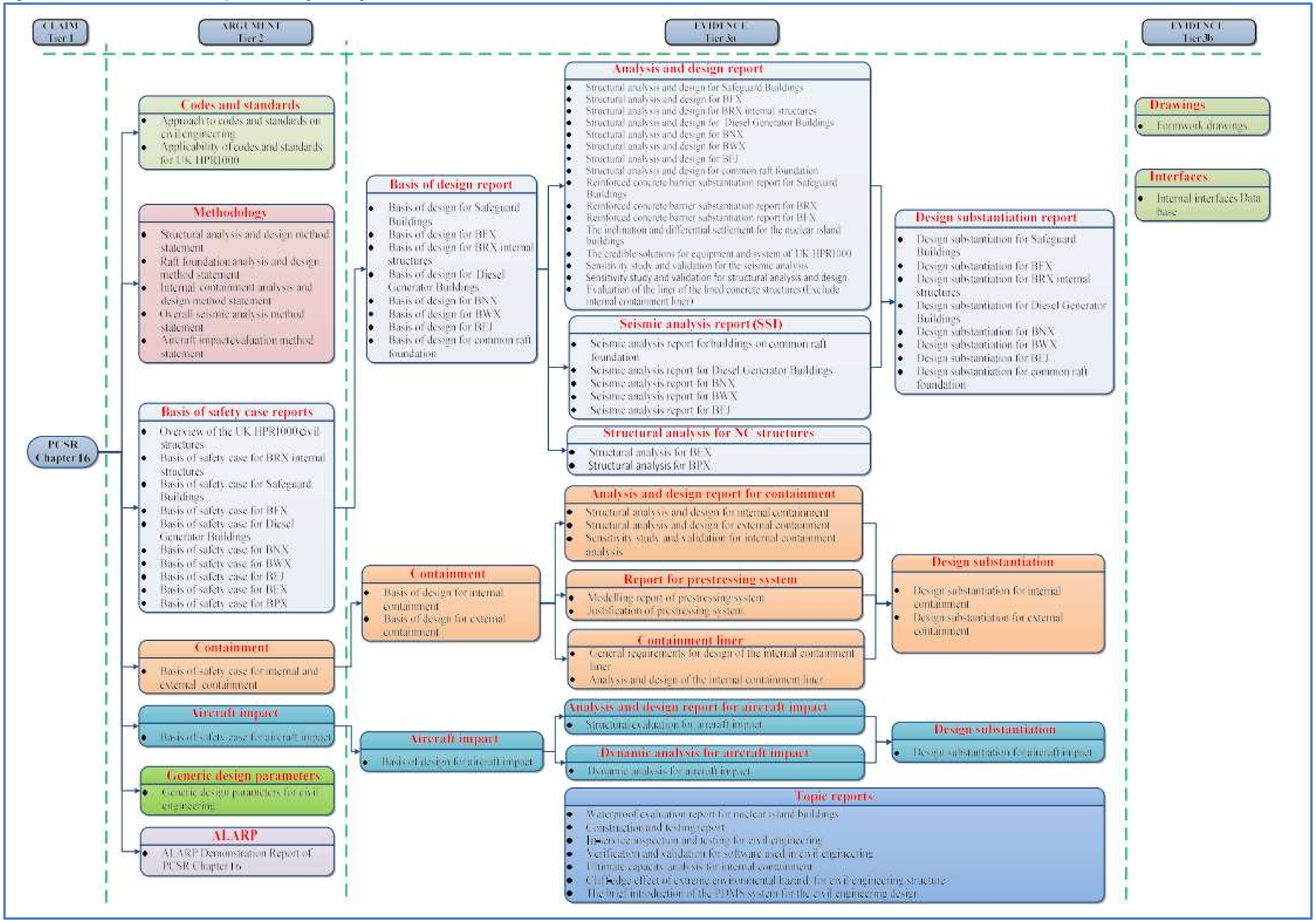


Figure 4: Structure of PCSR Chapter 16

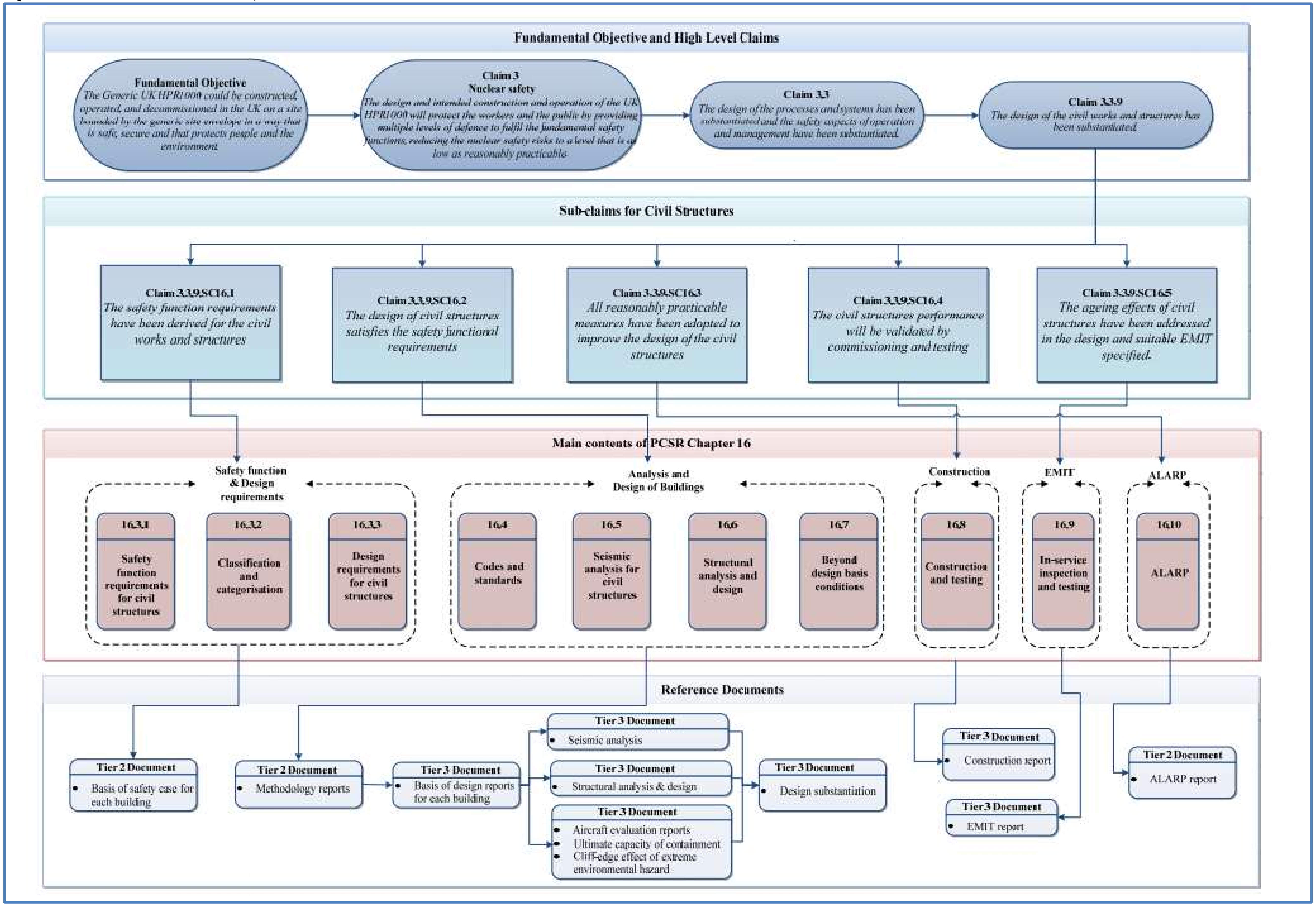


Figure 7: Diagram to show the RP's use of models in their analysis and design methodology