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Seismic Design Standards and Calculational Methods in the United States and Japan

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Seismic Design Standards and Calculational Methods in the United States and Japan

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ABSTRACT

Over the years, a number of nuclear power plants (NPPs) in Japan have experienced earthquake shaking and some have experienced shaking in multiple earthquakes. The U.S. Nuclear Regulatory Commission has identified a need to better understand the seismic performance of Japanese NPPs and to determine if any important lessons should be applied to NPPs in the United States (U.S.). Meeting that goal requires an understanding of the design criteria used in Japan, and the differences between the practices employed in the two countries.

This report provides information on current and past U.S. and Japanese seismic design standards, calculational methods, and load combinations used for the design of new and currently operating NPPs. This report documents the relative conservatism of the U.S. and Japan seismic analysis, design, and qualification inputs and processes. Both countries generally employ techniques that provide significant margin against the earthquake shaking levels used for design. This report covers various timeframes of interest for both countries.

This report provides information in the areas of seismic hazard assessment, classification categories, soil-structure interaction analyses, structural design, subsystem analysis and design, beyond-design-basis events, and seismic instrumentation. The report provides an assessment of the conservatisms in both the U.S. and Japanese approaches.

FOREWORD

On July 16, 2007, an earthquake occurred near the world's largest nuclear plant, the Kashiwazaki-Kariwa Nuclear Power Plant (KKNPP) located in the Niigata prefecture of Japan. The Niigataken Chūetsu-Oki (NCO) earthquake ground motion at the site exceeded the plant's seismic design-basis earthquake ground motion[1] and caused an extended period of shutdown of all seven reactors at the plant. Although the KKNPP units performed well given the exceedance of the design-basis ground motion that they experienced, the U.S. Nuclear Regulatory Commission (NRC) identified a need to understand and document the lessons learned from this earthquake.

To better understand the earthquake performance of the KKNPP units, the NRC initiated a multiphase project with the objective to develop, analyze, and document the impact of and lessons learned from the NCO earthquake on the KKNPP. This report addresses one particular aspect of the project by comparing U.S. and Japanese seismic design standards, calculational methods, and load combinations. The report documents the differences, similarities, and relative conservatisms of the U.S. and Japan seismic analysis, design, and qualification inputs and processes.

Although this report was developed in response to the 2007 NCO earthquake, itwill also provide an important resource for gaining an in-depth understanding of the tragic events that unfolded at the Fukushima Dai-ichi Nuclear Power Station that resulted from the March 2011 earthquake and tsunami as it related to the seismic performance of the plant's structures, systems, and components. However, this report does not provide an assessment of emergency response, severe accident mitigation strategies, or other defense-in-depth elements that may have a significant impact to the overall seismic risk of any particular facility, including Fukushima Dai-ichi and other NPPs impacted by the 2011 Tōhoku earthquake.

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[1] In Japan, the seismic design basis of safety-related structures, systems, and components (SSCs) is the envelope of the dynamic responses caused by the design-basis ground motions and applied equivalent static loading conditions. The NCO earthquake ground motion exceeded the design-basis ground motions by a significant amount, but the induced loading environment of the NCO earthquake on SSCs may not have exceeded the equivalent static loading conditions to the same degree.

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EXECUTIVE SUMMARY

Worldwide, the United States (U.S.) and Japan are two of the countries with the most developed seismic design standards and calculational methods for nuclear facilities with an emphasis on nuclear power plants (NPPs). Thousands of person-years of effort over 5 decades have been devoted to developing these standards and methods for nuclear facilities. In addition, Japan has experienced several earthquakes that have directly affected NPPs with ground motions exceeding the design-basis earthquake ground motions (denoted as S_1 or S_2) and in some cases significantly exceeding the S_2. In these cases, minimal or no damage from strong shaking of safety-related structures, systems, and components (SSCs) was observed. Even for the 2011 Tōhoku[1] earthquake, evidence suggests that damage of safety-related SSCs caused by strong shaking was not significant at any of the affected NPPs. Similarly, in the United States, a few events have occurred affecting NPPs, the most notable being the 2011 Mineral, VA, earthquake that caused ground motions at the North Anna Power Station that exceeded the plant's safe shutdown earthquake (SSE) ground motion over a large frequency range. Although the ground motion experienced at these U.S. sites exceeded the site-specific operating basis earthquake ground motion or SSE ground motion, the overall levels of shaking have been much lower than those experienced by NPPs in Japan. However, the same conclusion can be reached for the U.S. NPPs; minimal or no damage to safety-related SSCs was observed. This is a testament to the adequacy of the seismic design standards of the U.S. and Japan. Furthermore, field experiments and laboratory testing in the United States, Japan, and Taiwan over this same period have illuminated aspects of the standards and the conservatism contained therein. All of these factors have led to the evolution of seismic design standards and calculational methods over the last 5 decades. The lesson learned from the aggregate of these experiences is that seismic design of NPPs in the United States and Japan has been demonstrated to be extremely effective when tested by actual earthquake shaking.

In this context, a review of the state of practice in the United States and Japan is appropriate. The United States, Japan, and other countries can learn from these experiences and introduce appropriate changes to their seismic design standards and calculational methods. It is important to note, however, that seismic hazard assessment and seismic design must always be considered in the context of the seismo-tectonic environment in which the country exists. Nearly the entire country of Japan is situated in an area of high seismicity and both subduction and active crustal mechanisms are at work. The United States, by contrast, is highly varied with seismicity rates that range from very high to very low across its territory and with nearly every seismo-tectonic environment found within its borders. It should be expected, therefore, that differences in assessment, design, and regulatory approaches exist between the two countries.

On July 16, 2007, an earthquake occurred near the world's largest nuclear plant, the Kashiwazaki-Kariwa Nuclear Power Plant (KKNPP) located in the Niigata prefecture of Japan. The Niigataken Chūetsu-Oki (NCO) earthquake ground motion at the site exceeded the plant's seismic design-basis earthquake (DBE) ground motion by a significant amount[2] and caused an

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[1] The Tōhoku earthquake is formally called the Tohoku-chiho Taiheiyo-oki earthquake by the U.S. Geological Survey.

[2] In Japan, the seismic design basis of safety-related structures, systems, and components (SSCs) is the envelope of the dynamic responses caused by the design-basis ground motions and applied equivalent static loading conditions. The NCO earthquake ground motion exceeded the design-basis ground motions by a significant amount, but the induced loading environment of the NCO earthquake on SSCs may not have exceeded the equivalent static loading conditions to the same degree.

extended period of shutdown of all seven reactors at the plant. The KKNPP units generally performed well given the exceedance of the DBE ground motion that they experienced.

At the time of the March 11, 2011, Tōhoku earthquake, multiple units at KKNPP had been restarted. Restart of these units occurred after extensive evaluations of the effects of the NCO earthquake on SSCs, a reevaluation of the seismic DBE ground motion levels for the KKNPP Units 1 through 7, and confirmation that SSCs met the requirements for the new ground motion levels.[3] In some cases, modifications to existing SSCs were made to be consistent with Japanese design philosophy, e.g., ensuring high natural frequencies of subsystems.

As a result of the experiences at the KKNPP in 2007, the U.S. Nuclear Regulatory Commission (NRC) identified a need to understand and document the lessons learned from this earthquake. NRC staff has been working since the earthquake to collect lessons learned, to better understand differences between U.S. and Japanese design approaches, and to obtain quantitative data on plant performance during the event.

This NUREG/CR report is part of a multi-phase project with the objective to develop, analyze, and document the impact and lessons learned of the July 16, 2007 earthquake that affected the KKNPP in the Niigata prefecture of Japan. This report addresses the need to consider the similarities and differences in U.S. and Japanese seismic design standards and calculational methods, as well as load combinations.

Because both U.S. and Japanese standards have changed over time, specific time frames of interest are consistently used throughout the document. The time frames of applicability of the seismic design standards and methodologies are described in the following paragraphs:

- For the U.S., there are three time frames of interest when considering the seismic design-basis ground motion: 1973 to 1996, before the NRC issued Regulatory Guide (RG) 1.165, "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion," and enacted Title 10 of the *Code of Federal Regulations* (10 CFR) Part 50[4], "Domestic Licensing of Production and Utilization Facilities," Appendix S ("Earthquake Engineering Criteria for Nuclear Power Plants") in 1997; 1997 to 2007; and Post-2007, when NRC RG 1.208 ("A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion") was issued and numerous changes to other RGs and the Standard Review Plan took effect. "Pre-1997" refers to the seismic analysis and design procedures and requirements implemented for operating NPPs designed and constructed during this (pre-1997) period with emphasis on standards in place after about 1980. "Post-2007" refers to those standards currently in place and being applied to the next generation of NPPs in the United States. The Post-2007 standards are characterized by the use of Certified Designs. The period of 1997 to 2007 is discussed separately only with respect to the seismic design-basis earthquake. For purposes of this review, the periods of Pre-2007 and Post-2007 are generally meant to represent operating NPP designs (Pre-2007) and new NPP designs (Post-2007).

[3] As of February 2013, two nuclear power units in Japan are in operation (Ohi Power Station Units 3 and 4 operated by the Kansai Electric Power Co., Inc.). The remaining units in Japan are in suspension (some listed as undergoing periodic inspection) or in various stages of decommissioning (http://www.nsr.go.jp).

[4] It should be noted that the work supporting the 1973 regulatory changes had actually started in the1960s.

- For Japan, the timeframes of interest are denoted “Pre-2006” and “Post-2006.” “Pre-2006” refers to the seismic analysis and design procedures and requirements implemented for NPPs designed prior to 2006. “Post-2006” refers to the most recent change in criteria as stated by Japan’s Nuclear Safety Commission (on September 19, 2006). The new criteria are to be considered in the design of new facilities and the reevaluation of existing facilities. The seismic designs of the KKNPP units were performed for Pre-2006 criteria. Post-earthquake reevaluations of KKNPP units took into account Post-2006 requirements and guidelines.[5]

Seismic analysis, seismic design, and seismic qualification activities are multidisciplinary in nature and many elements comprise the analysis, design, qualification, and construction processes. Evaluating the relative conservatism in the individual elements is valuable but not necessarily indicative of the overall conservatism in one process compared to the other. One way to quantify differences in implementation of the seismic analysis, design, and qualification processes for the United States and Japan is to perform a pilot study, whereby a given design is compared step-by-step to quantify more or less conservatism in each step and in the final design. An abbreviated effort of this type is being performed for the International Atomic Energy Agency Extra-Budgetary Program Kashiwazaki-Kariwa Research Initiative for Seismic Margin Assessment (KARISMA) project, which is an additional phase of the NRC research project under which this report was developed. Information gained in the KARISMA project has been incorporated into this report, as appropriate.

Based on the assessment performed, the likely relative conservatisms for the operating reactors are summarized as follows:

Elements of the seismic analysis, design, and qualification processes for which Japan is more conservative than the U.S. (Japan Pre-2006 compared to U.S. Pre-2007)

- Structure damping values used in linear analysis are lower in Japan than in the United States.
- Damping values for some of equipment, components, and piping are specified to be lower in Japan than in the United States.
- Implementation, testing, and maintenance of modern seismic instrumentation systems are required in Japan and not in the United States.
- Testing for equipment seismic performance and fragility is performed in Japan, while proof testing is performed in the United States.

Elements of the seismic analysis, design, and qualification processes for which the U.S. is more conservative than Japan (Japan Pre-2006 compared to U.S. Pre-2007)

- All safety-related SSCs (Seismic Category I) are designed to SSE ground motion (by comparison, under the Japan criteria safety-related equipment is designed to S_1 and only a subset is assessed for functionality under the S_2 ground motion).
- Soil-structure-interaction (SSI) analyses to determine the structure response are performed for soil and soft rock sites. The structure response used for design, including as

[5] At the time of this writing, a new regulatory body has been formed in Japan and new codes and guidance is expected to be published in 2013 (see Section 2.2.2).

input to subsystems, is defined as the envelope of the responses for three soil profile cases in the United States; only a best estimate soil profile is considered in Japan.

- In-structure response spectra are developed with peaks broadened ±15 percent in the United States as compared to ±10 percent in Japan.
- Three components of earthquake ground motion are considered simultaneously in the SSI analyses.
- All combinations of loss-of-coolant accident (LOCA) loadings are combined with the SSE ground motions.
- Equipment qualification testing is required.
- Beyond design-basis ground motion evaluations are required for new plants; acceptance criteria for new plants, plant-level High Confidence of Low Probability of Failure (HCLPF) values must be greater than 1.67 times the design-basis ground motion. A seismic probabilistic risk assessment (SPRA) is required prior to loading of fuel in new reactors.
- For existing plants, similar (though early vintage) beyond design-basis assessment procedures were previously implemented and assessments performed during the Individual Plant Examination of External Events (IPEEE) program in the latter part of the 1990s. These assessment approaches subsequently matured and expanded into a number of tools that now exist for a variety of design, assessment, and operational uses. The current risk-informed performance-based operational and regulatory framework is a direct result of that early work and the lessons learned. The NRC is now in the process of re-assessing the seismic hazard for all U.S. operating reactors and will use the beyond design-basis tools for those NPPs whose new estimated ground motion exceeds the original design.

Elements of the seismic analysis, design, and qualification processes for which the relative conservatism is currently unknown (Japan Pre-2006 compared to U.S. Pre-2007)

- Probability of occurrence of peak values of design ground motion (peak ground acceleration (pga), peak ground velocity, and peak ground displacement) for the operating reactors is currently unknown, although analyses to determine this information are underway.
- In Japan, the maximum of static and dynamic loads are used for design (e.g., a static loading of 0.6g for structures and 0.72g for equipment, piping, etc.).
- In the United States, the minimum design ground motion at foundation level in the free-field has a minimum PGA of 0.1g anchoring a spectral shape appropriate for foundation level (outcrop or in-column motion). Most commonly a RG 1.60 (“Design Response Spectra for Seismic Design of Nuclear Power Plants”) spectrum is used.
- Automatic Seismic Trip Systems are required for all NPP units in Japan.

Elements of the seismic analysis, design, and qualification processes that are favorable for Japan in reducing uncertainty in dynamic behavior of SSCs and verifying seismic capacity

- Extensive testing program to verify behavior of soil-structure systems (SSI phenomena and methods of analysis).
- Extensive testing program to define the stiffness, nonlinear behavior, and capacity of structure elements, e.g., shear walls.

- Extensive testing program to define the behavior of equipment, piping, and other components.

Perspective

Historically, both the U.S. and the Japanese practices have used deterministic approaches in all aspects of the seismic analysis, design, and regulation. However, over the years, and particularly in connection with the new reactors, the U.S. practices are moving toward a more performance-based, risk-informed regulatory framework. The Japanese practice has recently begun to look at very limited aspects of risk-informed considerations. Its practice is still basically deterministic. The following describes how the risk-informed aspects are currently being used and provides a brief comparison of the two practices.

Japan has introduced the “residual risk” concept in 2006; however, the approach taken in seismic hazard assessment and seismic design is still inherently deterministic in nature. As in the most deterministic practices, the focus in Japan is on assuring that a high level of conservatism exists at every step in the design process, such that Japanese NPPs have significant margin above the DBE ground motion used. There is an assumption that the DBE ground motion used is sufficiently rare for the site of interest.

By contrast, the U.S. uses a mixed approach. For existing operating NPPs, meeting the NRC’s seismic-safety regulations still means meeting a complex set of deterministic regulations that are demonstrated by deterministic evaluations. This includes how the design-basis earthquake (the SSE) still in use was selected, although a probabilistic reevaluation of that SSE is now under way for all existing plants. For new designs, the same set of deterministic regulations, demonstrated by deterministic analyses, is still in place, except that the selection of the SSE for a new plant must follow a probabilistic seismic-hazard approach tied to a specific annual frequency of exceedance. What is new is that the regulatory evaluation of the design, which uses deterministic criteria similar to those used for the existing operating plants, is supplemented by a risk-informed and performance-based evaluation of the seismic adequacy of the plant-as-a-whole. This evaluation provides a clearer way to understand conservatisms inherent in the design and provides an opportunity to risk-inform the entire design practice.

These two philosophies are so different that the relative conservatism of the outcomes of the two approaches cannot be known *a priori*. The true conservatism of any regulatory framework for an NPP can only be assessed through a comparison of the true response of the NPP against the true hazard at its site. A seismic probabilistic risk assessment (SPRA) provides a means to evaluate the conservatisms.

Although for new plants the U.S. relies in part on a performance-based, risk-informed framework, the process of seismic analysis, seismic design, and seismic qualification of SSCs is deterministic by choice and the practicality of design. Deterministic procedures (methods and parameter values) are developed and evaluated to assure that the implementation of seismic analysis, seismic design, and seismic qualification for SSCs leads to SSC seismic performance that meets the risk guidelines.

A comparison of the results of the deterministic seismic analysis, design, and qualification process step-by-step is less appealing than a comparison of SPRA results; however, it is still a valuable exercise comparing the design loading conditions for SSCs, including loads, in-structure response spectra (ISRS) for qualification of equipment, components, and distribution systems, and other design quantities. This comparison could be conditional on the DBE or include the effects of the DBE. This approach quantifies the degree of relative conservatism introduced in various steps of the seismic analysis chain by U.S. procedures compared to the procedures of Japan. Results could also be interpreted in the risk framework as a surrogate for core damage frequency (CDF) or large early release frequency (LERF), such as onset of inelastic deformation. This is a very valuable and practical assessment process recognizing the multi-disciplinary nature of the process.

For the above reasons, the discussions in this document are framed to provide clarity and insights into the similarities and differences of the two regulatory approaches and frameworks. This document does not, and cannot, provide a strict “apples to apples” comparison of each step in the process.

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- Mr. Eiji Shirai, Committee Manager (Kansai Electric Power Company)
- Mr. Noriaki Tomura, Committee Member (Japan Atomic Power Company)
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[1] JANSI was formerly known as Japan Nuclear Technology Institute (also known as JANTI).

ABBREVIATIONS, ACRONYMS, UNITS, & PARAMETERS

III_AS	elastic stress state in JEAG 4601-1987
IV_AS	allowable stress state in JEAG 4601-1987
A	seismic classification category used in Japan (see Section 4.2)
A_s	seismic classification category used in Japan (see Section 4.2)
ACI	American Concrete Institute
AESJ	Atomic Energy Society of Japan
AIJ	Architectural Institute of Japan
AISC	American Institute of Steel Construction
ANS	American Nuclear Society
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASS	austenitic stainless steel
ASTS	automatic seismic trip system
B	seismic classification category used in Japan (see Section 4.2)
B_1	stress index taken from the ASME code (see Table 7-9)
B_2	stress index taken from the ASME code (see Table 7-9)
BE	best estimate
BWR	boiling-water reactor
C	seismic classification category used in Japan (see Section 4.2)
CAV	cumulative absolute velocity
CD	certified design
CDF	core damage frequency
CEUS	central and eastern United States
CFR	*Code of Federal Regulations*
C_I	story shear coefficient from JEAG 4601-1987
CLASSI	continuum linear analysis of soil-structure interaction
cm	centimeter
COL	construction and operating license
COV	covariance
CP	construction permit
C_s	stress limit designation in JAEC 4601-2008
CSDRS	Certified Seismic Design Response Spectra
C_V	vertical seismic coefficient from JEAG 4601-1987
D	dead load (see Table 7-14)
DAU	data acquisition unit
dB	decibels
DBE	design-basis earthquake ground motion
DC	design certification
DCPP	Diablo Canyon Power Plant
D_s	stress limit designation in JAEC 4601-2008
EBP	Extra-Budgetary Program
EPRI	Electric Power Research Institute
ESP	early site permit

f_b	allowable stress for bending (see Table 7-12)
f_c	allowable stress for compression (see Table 7-12)
FEM	finite element method
FOSID	frequency of the onset of significant inelastic deformation
f_s	allowable stress for shear (see Table 7-12)
f_t	allowable stress for tension (see Table 7-12)
f_p	allowable stress for bearing (see Table 7-12)
F_{ult}	ultimate strength
g	acceleration of gravity 9.81 m/s^2
GL	generic letter
GMPE	ground motion prediction equation
GMRS	Ground Motion Response Spectra
HCLPF	high confidence of low probability of failure
HNA	high-nickel alloy
HVAC	heating, ventilation, and air conditioning
HYDRO	transient or steady state suppression pool hydrodynamic loads (see Table 7-14)
Hz	Hertz
i	stress intensification factor
I&C	instrumentation and controls
IAEA	International Atomic Energy Agency
ISG	interim staff guidance
ISRS	in-structure response spectra
IEEE	Institute of Electrical and Electronics Engineers
In	inch
IPEEE	individual plant examination of external events
JANSI	Japan Nuclear Safety Institute
JEA	Japan Electric Association
JEAC	Japan Electric Association Code
JEAG	Japan Electric Association Guidelines
JMA	Japan Meteorological Agency
JNES	Japan Nuclear Energy Safety
JSME	Japan Society of Mechanical Engineering
KARISMA	Kashiwazaki-Kariwa Research Initiative for Seismic Margin Assessment
KKNPP	Kashiwazaki-Kariwa Nuclear Power Plant
km	kilometers
ksi	kips per square inch
LB	lower bound
LBB	leak-before-break
LERF	large early release frequency
LLNL	Lawrence Livermore National Laboratory
LOCA	loss-of-coolant accident or loss-of-coolant accident load
M	mechanical load in normal operating states (see Table 7-14)
m	meters

M_A	ASME dead weight moment in piping (see Table 7-9)
M_B	ASME seismic bending moments in piping (see Table 7-9)
M_D	mechanical load in operating state I or II or mechanical design load (see Table 7-14)
METI	Ministry of Economy and Trade and Industry (Japan)
M_{ip}	bending moment due to mechanical load including the earthquake inertia effects in JEAG 4601-1987 (see Table 7-9)
MITI	Ministry of International Trade and Industry (Japan)
M_L	nontransient mechanical load after LOCA (see Table 7-14)
mm/yr	millimeters per year
m/s	meters per second
NAPP	North Anna Power Plant
NCO	Niigataken Chūetsu-Oki (earthquake)
NEI	Nuclear Energy Institute (U.S.)
NISA	Nuclear and Industrial Safety Agency (Japan)
NPP	nuclear power plant
NRA	Nuclear Regulatory Authority (Japan)
NRC	Nuclear Regulatory Commission (U.S.)
NRO	New Reactors, Office of (NRC)
NRR	Nuclear Reactor Regulation, Office of (NRC)
NSC	Nuclear Safety Commission (Japan)
NSSS	nuclear steam supply system
NUREG	Nuclear Regulation Report (NRC)
OBE	operating basis earthquake ground motion
OL	operating license
P	normal operating pressure load (see Table 7-14)
P_D	mechanical pressure load from operating state I or II or maximum design pressure load (see Table 7-14)
PGA	peak ground acceleration
P_L	nontransient pressure load after LOCA
PRA	probabilistic risk assessment
PSA	probabilistic safety assessment (same as PRA)
PSHA	probabilistic seismic hazard analysis (or assessment)
PWR	pressurized-water reactor
Rem	Roentgen equivalent man (radiation unit)
RES	Nuclear Regulatory Research, Office of (NRC)
RG	Regulatory Guide
RRS	Required Response Spectrum
SASSI	system for analysis of soil-structure interaction
S	seismic classification category used in Japan (see Section 4.2)
S	allowable stress (JEAG Type 3 and 4 and ASME Class 2 and 3 components)
S_1	maximum design earthquake ground motion in Japan (see Section 3.2.1)
S_1*	envelope of S_1 dynamic loads and static loads
S_2	extreme design earthquake ground motion in Japan (see Section 3.2.1)
S_a	spectral acceleration
S_d	elastic design ground motion in Japan (see Section 3.2.2)

S_d*	envelope of S_d dynamic loads and static loads
SC	seismic category (used in U.S. terminology as SC-I, SC-II)
SDA	standard design approvals
SDAS	seismic data acquisition system
SECY	policy, rulemaking, and adjudicatory recommendation or informational paper issued by NRC staff to the Commission (through the Office of the Secretary)
SEI	Structural Engineering Institute
S_m	allowable stress for JEAG Type 1 and ASME Class 1 components
SMA	seismic margin assessment
SRM	staff requirements memoranda
SRP	standard review plan
SPRA	seismic probabilistic risk assessment
SPSA	seismic probabilistic safety assessment (same as SPRA)
SRSS	square root sum of the squares
S_s	design-basis ground motion in Japan (see Section 3.2.2)
SSC	structures, systems, and components
SSE	safe shutdown earthquake ground motion
SSHAC	Senior Seismic Hazard Analysis Committee
SSI	soil-structure interaction
S_u	ultimate tensile strength
S_{ult}	ultimate strength
S_y	yield strength
TEPCO	Tokyo Electric Power Company
TRS	Test Response Spectra
UB	upper bound
UHRS	uniform hazard response spectrum
U.S.	United States
V_s	shear wave velocity
WUS	western United States
yr	year

1 INTRODUCTION

1.1 Background

On July 16, 2007, an earthquake occurred near the world's largest nuclear plant, the Kashiwazaki-Kariwa Nuclear Power Plant (KKNPP) located in the Niigata prefecture of Japan. KKNPP is owned and operated by the Tokyo Electric Power Company (TEPCO). The Niigataken Chūetsu-Oki (NCO) earthquake ground motion at the site exceeded the plant's seismic design basis and caused an extended period of shutdown of all seven reactors at the plant. Units 1, 5, 6, and 7 were restarted between May 2009 and November 2010. Restart of these units occurred after TEPCO's extensive evaluations of the effects of the NCO earthquake on the plant's structures, systems, and components (SSCs), a reevaluation of the seismic ground motions for KKNPP units 1 through 7, and confirmation that the SSCs met the requirements for the new ground motion levels. In some cases, modifications to existing SSCs were made to be consistent with Japanese design philosophy, e.g., ensuring high natural frequencies of subsystems. At the time of the March 11, 2011, Tōhoku earthquake, units 2, 3, and 4 were still under investigation by the local government[1].

After the earthquake, the United States (U.S.) Nuclear Regulatory Commission (NRC) identified a need to understand and document the lessons learned from the event. NRC staff has been working since the earthquake to collect lessons learned, to better understand differences between U.S. and Japanese design approaches, and to obtain quantitative data on plant performance during the event. In this regard, NRC staff has been working both in-house and through international avenues, such as by meeting with Japanese colleagues and through involvement with the activities of the International Atomic Energy Agency (IAEA). As part of the effort to analyze this event, the NRC participated in an IAEA Extra-Budgetary Program (EBP) on Seismic Issues. Under this EBP, several initiatives were undertaken and a significant amount of information was made available to NRC staff concerning the design criteria and performance of the KKNPP and other Japanese nuclear power plants (NPPs). This information was provided by Japan Nuclear Energy Safety Organization, Tokyo Electric Power Company, and the Nuclear and Industrial Safety Agency. This international interaction was critical because most Japanese standards and regulatory guidance documents are available only in Japanese. It was the consensus of the international participants of the IAEA EBP that the KKNPP units performed well given the exceedance of the design-basis ground motion that they experienced. However, it was noted that similar performance would not be assured, or necessarily expected, for NPPs elsewhere given the same loading conditions (or relative loading conditions) due to differing design standards.

Seismic hazard assessment and seismic design must always be considered in the context of the seismo-tectonic environment in which the country exists. Nearly the entire country of Japan is situated in an area of high seismicity and both subduction and active crustal mechanisms are at work. The U.S., in contrast, is highly varied with seismicity rates that range from very high to very low across its territory and with nearly every seismo-tectonic environment found within its borders. It should be expected, therefore, that differences in assessment, design, and regulatory approaches exist between the two countries.

[1] In the regulatory structure in Japan, local governments must agree to allow restart before restart can occur.

1.2 Purpose

To better understand the earthquake performance of the KKNPP units, particularly as it relates to U.S. designs, the NRC initiated a multi-phase project to develop, analyze and document the impact and lessons learned of the July 16, 2007, earthquake that affected the KKNPP in the Niigata prefecture of Japan. This multi-phase project includes the following:

1. Summarize existing information on the impacts of the earthquake on the facility and lessons learned to-date ("Summary of Information on the Effects of the Niigataken Chūetsu-Oki Earthquake on the Kashiwazaki-Kariwa Nuclear Power Plant" - ML15342A306)[2]

2. Summarize the similarities and differences in U.S. and Japanese seismic design standards, calculational methods, and load combinations.

3. Assess the performance of the KKNPP in light of Japanese seismic design standards; summarize known lessons learned as identified by Japanese, IAEA, and NRC staff; and review possible implications of the lessons learned on U.S. plants and processes ("Impacts of the Niigataken Chūetsu-Oki Earthquake on the Kashiwazaki-Kariwa Nuclear Power Plant, Post-Earthquake Response, and Lessons Learned" - ML15342A311)[3]

4. Participate in the Kashiwazaki-Kariwa Research Initiative for Seismic Margin Assessment (KARISMA) benchmarking exercise of the IAEA EBP on Seismic Issues to assess the performance of U.S. modeling approaches as compared to other member states and the actual behavior of the KKNPP. The benchmark is a multi-phase project. Phase 1 entailed modeling the soil in the neighborhood of the KKNPP nits 5, 6, and 7 for low strain behavior, behavior during the aftershocks, and the behavior during the main shock, modeling the wave propagation characteristics in the free-field for the same conditions, developing a dynamic structure model of the KKNPP Unit 7 Reactor Building, and constructing a soil-structure interaction model for the KKNPP Unit 7 Reactor Building. Phase 2 focused on the analysis of the Unit 7 Reactor Building subjected to the NCO earthquake ground motion. Phase 3 involved evaluating seismic margins in the structure, system, and component as evidenced in additional analyses and documenting the findings("Impacts of the Niigataken Chūetsu-Oki Earthquake to the Kashiwazaki-Kariwa Nuclear Power Plant, Post-Earthquake Response, and Lessons Learned: U.S. Perspective" ML15342A314)[4]

This NUREG report is Phase 2 and provides a discussion of U.S. and Japan seismic design standards, calculational methods, and load combinations. Throughout this report, the time frames of applicability of the seismic design standards and methodologies are delineated:

- For the U.S., there are three time frames of interest when considering the seismic design-basis earthquake: 1973[5] to 1996 (this period ended in 1997 when the NRC issued

[2] Report for for internal use only due to prorietary information and cannot be released to the public.

[3] Report for for internal use only due to prorietary information and cannot be released to the public.

[4] Report for for internal use only due to prorietary information and cannot be released to the public.

[5] Prior to 1973, seismic design followed the standard building code. As a result, later seismic reassessment programs were developed to address the early design of US NPPs. For additional discussion of NRC seismic requirements over time, please see [NRC 2011]

Regulatory Guide (RG) 1.165, “Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion,” and Title 10 of the *Code of Federal Regulations* (10 CFR) Part 50, “Domestic Licensing of Production and Utilization Facilities,” Appendix S (“Earthquake Engineering Criteria for Nuclear Power Plants”) was enacted; 1997 to 2007; and Post-2007. In 2007, NRC RG 1.208 (“A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion”) was enacted and numerous changes to other RGs and the Standard Review Plan took effect.

“Pre-1997” refers to the seismic analysis and design procedures and requirements implemented for operating NPPs designed and constructed during this (pre-1997) period with emphasis on standards in place after about 1980. “Post-2007” refers to those standards currently in place and being applied to the next generation of NPPs in the United States. The new regulatory framework is characterized by the use of Certified Designs. The period of 1997 to 2007 is discussed separately only with respect to the seismic design-basis earthquake. For purposes of this review, the periods of Pre-2007 and Post-2007 are generally meant to represent currently operating NPP designs (Pre-2007) and new NPP designs (Post-2007).

- For Japan, the time frames of interest are denoted as “Pre-2006” and “Post-2006.” Pre-2006 refers to the seismic analysis and design procedures and requirements implemented for NPPs designed before 2006. Post-2006 refers to the most recent change in criteria as stated by Japan’s Nuclear Safety Commission (September 19, 2006). These most recent criteria are to be considered in the design of new facilities and the reevaluation of existing facilities.

The seismic designs of the KKNPP units were performed using Pre-2006 criteria. Post-earthquake reevaluation of KKNPP units took into account Post-2006 requirements and guidelines.

1.3 Structure of the Report

Section 2 presents a historical perspective on U.S. and Japanese seismic design criteria development and the bases for the discussions contained in subsequent sections. Section 3 contains a discussion of the design-basis earthquake development for the U.S. and Japan. Section 4 presents a discussion of seismic design categorization. Section 5 compares structural response methodologies for the U.S. and Japan. Section 6 compares elements of structural design. Section 7 discusses subsystem response, design, and qualification approaches. Section 8 discusses beyond design-basis ground motion considerations. Section 9 presents information on seismic instrumentation. Section 10 contains overall summaries and conclusions. Section 11 contains the list of references.

1.4 Perspective on the Comparison

Historically, both the U.S. and the Japanese practices have used deterministic approaches in all aspects of the seismic analysis, design, and regulation. However, over the years, and particularly in connection with the new reactors, the U.S. practices are moving toward a more performance-based, risk-informed regulatory framework. The Japanese practice has recently begun to look at very limited aspects of risk-informed considerations. Its practice is still basically deterministic. The following describes how the risk-informed aspects are currently being used and provides a brief comparison of the two practices.

Japan introduced the “residual risk” concept in 2006; however, the approach taken in seismic hazard assessment and seismic design is still inherently deterministic in nature. As in most deterministic practices, the focus in Japan is on assuring that a high level of conservatism exists at every step in the design process, such that Japanese NPPs have significant margin above the design-basis earthquake (DBE) ground motion used. There is an assumption that the DBE ground motion used is sufficiently rare for the site of interest.

By contrast, the U.S. uses a mixed approach. For existing operating NPPs, meeting the NRC's seismic-safety regulations still means meeting a complex set of deterministic regulations that are demonstrated by deterministic evaluations. This includes how the design-basis earthquake (the Safe Shutdown Earthquake (SSE)) still in use was selected, although a probabilistic reevaluation of that SSE is now under way for all existing plants. For new designs, the same set of deterministic regulations, demonstrated by deterministic analyses, is still in place, except that the selection of the SSE for a new plant must follow a probabilistic seismic-hazard approach tied to a specific annual frequency of exceedance. What is new is that the regulatory evaluation of the design, which uses deterministic criteria similar to those used for the existing operating plants, is supplemented by a risk-informed and performance-based evaluation of the seismic adequacy of the plant-as-a-whole. This evaluation provides a clearer way to understand conservatisms inherent in the design and provides an opportunity to risk-inform the entire design practice.

These two philosophies are so different that the relative conservatism of the outcomes of the two approaches cannot be known *a priori*. The conservatism of any regulatory framework for an NPP can only be assessed through a comparison of the true response of the NPP against the true hazard at its site. A seismic probabilistic risk assessment provides a means to evaluate the conservatisms.

Although for new plants the U.S. relies in part on a performance-based, risk-informed framework, the process of seismic analysis, seismic design, and seismic qualification of structures, systems, and components (SSCs) is deterministic by choice and the practicality of design. Deterministic procedures (methods and parameter values) are developed and evaluated to assure that the implementation of seismic analysis, seismic design, and seismic qualification for SSCs leads to SSC seismic performance that meets the risk guidelines.

A comparison of the results of the deterministic seismic analysis, design, and qualification process step-by-step is less satisfying than a comparison of seismic probabilistic risk assessment (SPRA) results; however, it is still a valuable exercise. The end result is a comparison of the design loading conditions for SSCs, including loads, in-structure response spectra (ISRS) for qualification of equipment, components, and distribution systems, and other design quantities. This comparison could be conditional on the DBE or include the effects of the DBE. The end result quantifies the degree of relative conservatism introduced in various steps of the seismic analysis chain in U.S. procedures compared to the procedures of Japan. The end result could also be interpreted in the risk framework as a surrogate for core damage frequency (CDF) or large early release frequency (LERF), such as onset of inelastic deformation. This is a very valuable and practical assessment process recognizing the multi-disciplinary nature of the process.

For the above reasons, the discussions in this document are framed to provide clarity and insights into the similarities and differences of the two regulatory approaches and frameworks. This document does not, and cannot, provide a strict "apples to apples" comparison of each step in the process.

2 SEISMIC DESIGN CRITERIA FRAMEWORK, DEVELOPMENT, AND BASES OF COMPARISON

The basic principle underpinning both the Japanese and United States (U.S.) seismic design practices is that s*tructures, systems, and components (SSCs) important to safety are required to withstand the effects of earthquakes, and perform their required functions*.

2.1 Seismic Design Criteria Development, Regulatory Approach, and Framework in the United States

The first nuclear reactors in the United States were designed to the seismic requirements that existed for commercial facilities at that time. A concerted effort was made in the late 1960s and early 1970s to develop more stringent design standards for nuclear power plants (NPPs) in general, including provisions for seismic design. As part of this effort, new U.S. pressure vessel, piping, pump, and valve standards were derived from the U.S. Navy nuclear submarine program. Standards for qualification of electrical, instrumentation, and control equipment were developed by the Institute of Electrical and Electronics Engineers (IEEE). Passive equipment, other than the pressure vessel and piping, was typically designed to industrial standards for steel structures with modifications for high-level seismic demand.

2.1.1 Laws and Acts

The Energy Reorganization Act of 1974 is the fundamental U.S. law on both civilian and military uses of nuclear material. On the civilian side, the act provides for the development and regulation of the uses of nuclear materials and facilities in the United States. The functions of civilian development and promotion were split from civilian regulation. The Department of Energy was made responsible for the development and promotion of civilian nuclear power as well as development of nuclear weapons. The Nuclear Regulatory Commission (NRC) was assigned the responsibility for regulations. NRC's jurisdiction is solely the regulation of civilian use of nuclear materials and does not include regulation of defense nuclear facilities or promotion activities.

2.1.1.1 Founding of the U.S. Nuclear Regulatory Commission

The U.S. NRC, formerly the Atomic Energy Commission, was established by the Energy Reorganization Act in 1974. The NRC is an independent regulatory body that is responsible for assuring that the requirements of Title 10 of the *Code of Federal Regulations* (CFR) are carried out.

The NRC is headed by a five-member Commission. The Commissioners are appointed by the President and approved by the U.S. Senate. Consistent with the CFR, the Commission formulates policies and regulation governing nuclear reactor and material safety, issues orders to licensees, and adjudicates legal matters brought before it. An Advisory Committee on Reactor Safeguards has statutory responsibilities as described in the Atomic Energy Act of 1954 on matters referred to it by the Commissioners. The NRC staff carries out the policies and regulatory requirements of the Commission.

2.1.1.2 Code of Federal Regulations

Title 10 of the CFR consists of many parts with legal requirements. The parts that contain regulations relative to seismic design are:

- 10 CFR Part 50, “Domestic Licensing of Production and Utilization Facilities”
- 10 CFR Part 100, “Reactor Site Criteria”
- 10 CFR Part 52, “Licenses, Certifications, and Approvals for Nuclear Power Plants”

NPP general design criteria are governed by regulations in 10 CFR 50, Appendix A, “General Design Criteria for Nuclear Power Plants.” As of today, Appendix A contains 64 general criteria that are required to be met by U.S. law. Five overall requirements include General Design Criterion 2, “Design Bases for Protection against Natural Phenomena,” which states, in part, “Structures, systems, and components important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function.”

Regulations in 10 CFR Part 100 are associated with siting of NPPs. Appendix A to 10 CFR Part 100, “Seismic and Geologic Siting Criteria for Nuclear Power Plants,” issued in 1973, codified geologic and seismic regulatory practice that had evolved from the inception of nuclear power generation to that time. NPPs licensed prior to January 1997 comply with the seismic siting requirements of Appendix A to 10 CFR Part 100. Those licensed after January 1997 must comply with the geologic and seismic siting criteria of 10 CFR 100.23, “Geologic and Seismic Siting Criteria,” and the seismic design requirements of Appendix S to 10 CFR Part 50, “Earthquake Engineering Criteria for Nuclear Power Plants.”

The movement to Certified Designs is governed by 10 CFR Part 52, “Licenses, Certifications, and Approvals for Nuclear Power Plants.” As stated in 10 CFR Part 52, “This part governs the issuance of early site permits, standard design certifications, combined licenses, standard design approvals, and manufacturing licenses for nuclear power facilities.…”

2.1.2 Licensing Framework

The Licensing process has historically been a two-step process:

- Construction Permit (CP)
- Operating License (OL)

The new system, described in 10 CFR Part 52, is composed of either two or three elements:

- Design Certification (DC)
- Early Site Permit (ESP)
- Combined Construction and Operating License (COL)

The design certification process is intended to provide a standard nuclear island design that is certified for a site independent earthquake ground motion. The ESP addresses the site-specific seismic hazard and other hazards at a chosen site. It is not required, but can be referenced in the COL and can be beneficial in some situations. In the COL, the same process is followed as the previous policy (10 CFR Part 50) except public input is restricted to construction and operation issues provided that the DC (and possibly ESP) has already been granted. Safety-related structures and subsystems outside of the nuclear island are not included in the DC and are the responsibility of the COL applicant.

2.1.3 U.S. NRC Regulatory Guides

The NRC issues Regulatory Guides (RGs) that provide technical “guidance to licensees and applicants on implementing specific parts of the NRC regulations, techniques used by the NRC staff in evaluating specific problems or postulated accidents, and data needed by the staff in its review of applications for permits or licenses.”

(http://www.nrc.gov/reading-rm/doc-collections/reg-guides/) RGs are not substitutes for regulations, and compliance with them is not required. Methods and solutions that differ from those set forth in RGs may be deemed acceptable, if the applicant provides acceptable bases for the approach as required for the issuance or continuance of a permit or license by the NRC.

The U.S. NRC staff has developed a series of RGs and a Standard Review Plan (SRP, also known as NUREG-0800) [NRC 2007a] and develops regulatory documents published as NUREG series reports that provide the technical basis for regulatory actions.

There are 10 divisions of RGs. Division 1 contains guides for power reactors. The SRP (discussed in the next section) is a comprehensive document that covers all technical aspects of siting, design, and safety analysis of power reactors. It is intended as a guide for NRC staff members who review submittals by licensees; however, in practical terms, it is also used by the industry as a guidance document. Important Division 1 RGs that affect seismic design of structures and subsystems are[1]:

- RG 1.12, “Nuclear Power Plant Instrumentation for Earthquakes,” Revision 2 [NRC 1997a]
- RG 1.29, “Seismic Design Classification,” Revision 4 [NRC 2007b]
- RG 1.60, “Design Response Spectra,” Revision 1 [NRC 1973] (generally superseded for new plants, but a historically significant document for existing plants)
- RG 1.61, “Damping Values,” Revision 1 [NRC 2007c]
- RG 1.84, “Design Fabrication and Materials Code Case Acceptability,” *American Society of Mechanical Engineers (ASME) Section III*, Revision 33 [NRC 2005]
- RG 1.92, “Combining Modal Responses and Spatial Components in Seismic Response Analysis,” Revision 2 [NRC 2006a]
- RG 1.100, “Seismic Qualification of Electrical and Mechanical Equipment,” Revision 3 [NRC 2009a]
- RG 1.122, “Development of Floor Design Response Spectra for Seismic Design of Floor-Supported Equipment of Components,” Revision 1 [NRC 1978]
- RG 1.124, “Service Limits and Loading Combinations for Class 1 Linear-Type Supports,” Revision 2 [NRC 2007d]

[1] Regulatory Guide revisions cuurent as of the beginning of this report project and may not be the current revision

- RG 1.130, “Service Limits and Loading Combinations for Class 1 Plate and Shell-Type Component Supports,” Revision 2 [NRC 2007e]
- RG 1.136, “Design Limits, Loading Combinations, Materials, Construction and Testing of Concrete Containments,” Revision 3 [NRC 2007f]
- RG 1.142, “Safety-Related Concrete Structures for Nuclear Power Plants,” Revision 2 [NRC 2001a]
- RG 1.165, “Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion,” [NRC 1997c], (withdrawn April 30, 2010)
- RG 1.166, “Pre-Earthquake Planning and Immediate Nuclear Power Plant Operator Post-Earthquake Actions” [NRC 1997d]
- RG 1.167, “Restart of a Nuclear Power Plant Shut Down by a Seismic Event” [NRC 1997e]
- RG 1.174, “An Approach for Using Probabilistic Risk Assessment in Risk-Informed Decisions on Plant-Specific Changes to the Licensing Basis” [NRC 2002]
- RG 1.193, “ASME Code Cases not Approved for Use,” Revision 2 [NRC 2007g]
- RG 1.199, “Anchoring Components and Structural Supports in Concrete” [NRC 2003]
- RG1.200, “An Approach for Determining the Technical Adequacy of Probabilistic Risk Assessment Results for Risk-Informed Activities” [NRC 2009b]
- RG 1.201, “Guidelines for Categorizing Structures, Systems and Component in Nuclear Power Plants According to Their Safety Significance,” Revision 1 [NRC 2006b]
- RG 1.206, “Combined License Applications for Nuclear Power Plants (Light Water Reactor Edition)” [NRC 2007h]
- RG 1.208, “A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion” [NRC 2007i]

Until April 30, 2010, both RG 1.165 and RG 1.208 were valid RGs. Both are discussed in this document. However, RG 1.165 was withdrawn and superseded by RG 1.208 effective April 30, 2010. The withdrawal states that such withdrawal does not affect the licensing basis of any currently operating reactor or any of the currently issued early site permits under 10 CFR Part 52, subpart A. Additional clarifications are stated relative to approved Design Certifications and those in review as of April 30, 2010, that used RG 1.165 as a basis. At the time of that writing (2010), the U.S. NRC staff expected new applicants to use RG 1.208, which is performance-based (i.e., the performance of NPP structures are considered in the development of design response spectra) and requires the use of probabilistic seismic hazard assessment methods.

2.1.4 U.S. NRC Standard Review Plan

The SRP for the Review of Safety Analysis Reports for Nuclear Power Plants (NUREG-0800) provides guidance to NRC staff in performing safety reviews of construction permit (CP) or operating license (OL) applications (including requests for amendments) under 10 CFR Part 50 and ESP, DC, COL, standard design approval (SDA), or manufacturing license applications under 10 CFR Part 52 (including requests for amendments). “The principal purpose of the SRP is to assure the quality and uniformity of staff safety reviews. It is also the intent of this plan to make information about regulatory matters widely available and to improve communication between the

NRC, interested members of the public, and the nuclear power industry, thereby increasing understanding of the NRC's review process." (Introduction, NUREG-0800) Compliance with the SRP is not required, but meeting its conditions increases the likelihood of acceptance by NRC staff (not complying with the SRP requires justification). The SRP is intended as a staff guide but is typically utilized in the nuclear industry as a guidance document.

Sections of the SRP that pertain to seismic design are:

- 2.5.2 Vibratory Ground Motion
- 2.5.3 Surface Faulting
- 2.5.4 Stability of Subsurface Material and Foundations
- 2.5.5 Stability of Slopes
- 3.2.1 Seismic Classification
- 3.7.1 Seismic Design Parameters
- 3.7.2 Seismic Systems Analysis
- 3.7.3 Seismic Subsystems Analysis
- 3.7.4 Seismic Instrumentation
- 3.8.1 Concrete Containment
- 3.8.2 Steel Containment
- 3.8.3 Concrete and Steel Internal Structures of Steel or Concrete Containments
- 3.8.4 Other Seismic Category 1 Structures
- 3.8.5 Foundations
- 3.9.2 Dynamic Testing and Analysis of Systems, Structures and Components
- 3.9.3 ASME Class 1,2, and 3 Components, Component Supports and Core Support Structures
- 3.9.4 Control Rod Drive Systems
- 3.9.5 Reactor Pressure Vessel Internals
- 3.9.6 Functional Design, Qualification and In Service Testing Programs for Pumps, Valves and Dynamic Restraints
- 3.10 Seismic and Dynamic Qualification of Mechanical and Electrical Equipment
- 3.12 ASME Code Class 1, 2 and 3 Piping Systems, Piping Components and Their Associated Supports
- 3.13 Threaded Fasteners - ASME Code Class 1, 2 and 3
- 19.0 Probabilistic Risk Assessment and Severe Accident Evaluation for New Reactors

At the time of this writing (2013), revisions to SRP Sections 3.7 and 3.8 are proposed.[2]

2.1.5 Other Important Regulatory Documents

Other important regulatory documents include Branch Technical Positions and Interim Staff Guidance (ISG) documents prepared by the staff to provide review guidance where RGs or the SRP have not yet been formalized for the specific issues. Of particular interest are the following ISGs:

- U.S. NRC, "Interim Staff Guidance on Seismic Issues Associated with High Frequency Ground Motion in Design Certification and Combined License Applications," DC/COL-ISG-1 [NRC 2008a]
- U.S. NRC, "Interim Staff Guidance, Probabilistic Risk Assessment Information to Support Design Certification and Combined License Applications," DC/COL-ISG-003 [NRC 2008b]
- U.S. NRC, "Interim Staff Guidance on Ensuring Hazard-Consistent Seismic Input for Site Response and Soil-Structure Interaction Analyses," DC/COL-ISG-017 [NRC 2010a]
- U.S. NRC, "Interim Staff Guidance on Implementation of a Probabilistic Risk Assessment-Based Seismic Margin Analysis for New Reactors," DC/COL-ISG-020 [NRC 2010b]

There are also policy issue documents, developed by NRC staff and that contain positions recommended to the five-member Commission, that are known as "SECYs". The Commission may approve or modify the recommended positions by issuing a staff requirements memoranda (SRM) document detailing the Commission's policy directive. In SECY-93-087 [NRC 1993], "Policy, Technical and Licensing Issues Pertaining to Evolutionary and Advanced Light-Water Reactor Designs," the Commission approved the staff recommendation that the plant designer perform a Probabilistic Risk Assessment-based Seismic Margin Assessment (PRA-based SMA). The staff recommended that the resulting plant-level High Confidence in Low Probability of Failure (HCLPF) value should be at least twice the design ground motion peak ground acceleration and associated design ground response spectra. The Commission did not approve the factor of 2 for plant level HCLPF but approved a value of 1.67 times the design-basis safe shutdown earthquake (SSE) ground motion for the margin assessment. Industry has taken the position that this value will be met due to the inherent conservatism in the seismic design process. To date, the design certification stage of licensing for new reactors in the United States has mostly used generic values of fragilities for all but the reactor-building complex and the generic values all have HCLPF values greater than the target plant-level HCLPF value of 1.67 times the SSE ground motion.

While the target HCLPF level will generally be achieved for the plant-level and for individual components, there is no guarantee that meeting the minimum requirements of the applicable codes and standards will result in a HCLPF that is 1.67 times the SSE ground motion for any specific SSC and the values typically have not been confirmed through SSC-specific testing or non-linear analyses performed for beyond design-basis loading levels. Examples where there may be marginal HCLPFs include fuel assemblies and steam generator upper supports. Also, for components that are qualified by testing, there is only a 10 percent factor of safety applied to the required response spectrum (RRS) and, unless there is significant over-testing, or unless the RRS

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[2] See Memorandum from Anna H. Bradford, Acting Chief, Policy Branch, Division of Advanced Reactors and Rulemaking, Office of New Reactors, to Edwin M. Hackett, Executive Director, Advisory Committee on Reactor Safeguards and Richard P. Correia, Chairman, Committee to Review Generic Requirements, "Issuance of Draft Revision(s) to Standard Review Plan Section(s) – See List Below," March 28, 2013 (Agencywide Documents Access and Management System (ADAMS) Accession No. ML13086A185).

are very conservatively defined, the HCLPF for function of the equipment during the earthquake may not meet the 1.67 times SSE criterion for a site where the SSE ground motion levels are close to the certified seismic design response spectrum (CSDRS).

To address this issue, the American Society of Civil Engineers/Structural Engineering Institute standard ASCE/SEI 43-05 [ASCE/SEI 2005] recommends that a load factor of 1.4 be applied to the required response spectra. This recommendation is not included in the latest 2004 version of IEEE-344 [IEEE 2004] or in the current RG 1.100, both of which are deterministic in nature. However, this factor is being reviewed in the NRC's seismic research program for inclusion in future revisions of RG 1.100, which are expected to include probabilistic methods.

2.1.6 Professional Society Codes and Standards

Details of structural and subsystem design are contained in national standards that are endorsed by the NRC. The current codes and standards (developed by ASCE, SEI, American Society of Mechanical Engineers (ASME), American Concrete Institute (ACI), American National Standards Institute (ANSI), American Institute of Steel Construction (AISC) and IEEE and endorsed by the NRC) are:

- ASCE/SEI 43-05, "Seismic Design for Structures, Systems and Components in Nuclear Facilities," American Society of Civil Engineers/Structural Engineering Institute, 2005 [ASCE/SEI 2005] (ASCE 43-05 is not endorsed in total by the NRC.)
- ASCE 4-98, "Seismic Analysis of Safety-Related Nuclear Structures and Commentary," American Society of Civil Engineers, 2000 [ASCE 1998] (ASCE 4-98 is not endorsed in total by NRC). A revision to ASCE 4 in currently in progress and is expected to be published in 2013.
- ASME Boiler and Pressure Vessel Code, Section III, Division 1, "Nuclear Power Plant Components for Class 1, 2 and 3 pressure vessels, pumps, valves, piping and supports (Subsections NA, NB, NC, ND and NF), core support structures (Subsection NG) and steel containment (Subsection NE)," [ASME 2013a and as described in NUREG 0-800]
- ASME Boiler and Pressure Vessel Code, Section III, Division 2 for "Construction of Concrete Reactor Vessels and Containment," [ASME 2013b and as described in NRC NUREG 0-800]
- ACI 349, "Code Requirements for Nuclear Safety Related Concrete Structures" [ACI 2007b]
- ACI 359-07, "Code for Concrete Containments, American Concrete Institute, American Concrete Institute," July 2007. (This document forms Division 2 of the ASME Boiler and Pressure Vessel Code, Section III.) [ACI 2007a]
- ANSI/AISC N-690 1994 Including Supplement 2, "Specification for Safety Related Structures for Design, Fabrication and Erection of Steel Structures for Nuclear Facilities" [ANSI/AISC 1994]
- IEEE 344, "IEEE Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations" [IEEE 2004]
- IEEE 323, "Standard for Qualifying Class 1E Equipment for Nuclear Power Generating Stations" [IEEE 2003]
- IEEE C37.98, "IEEE Standard for Seismic Testing of Relays" [IEEE 1987]

2.1.7 Industry Technical Reports

The nuclear industry, led by the Electric Power Research Institute (EPRI), provides valuable research on specific issues and interacts with the U.S. NRC, directly with the NRC Office of Nuclear Regulatory Research (RES), or through the Nuclear Energy Institute with the NRC Office of Nuclear Reactor Regulation (NRR) and the NRC Office of New Reactors (NRO), on behalf of the industry to resolve particular concerns. In 2006 and 2007, a number of important seismic issues were addressed including:

- Definition of the lower bound of earthquakes to be considered in Probabilistic Seismic Hazard Assessments (PSHAs) by minimum cumulative absolute velocity values [EPRI 2006a]
- Ground motion attenuation model uncertainty and truncation of ground motions [EPRI 2006b]
- Ground motion coherence functions based on recorded data for addressing high frequency ground motion issues [EPRI 2007d]
- Soil-structure interaction (SSI) analysis procedures to model effects of incoherence of ground motion on structures [EPRI 2007c]
- Approach to verify that high frequency excitations do not adversely affect equipment and components [EPRI 2007a and EPRI 2007b]

As discussed in Section 1, the U.S. criteria are reviewed for the Pre-2007 and Post-2007 time frames. The above listings of reference documents show the most recent revisions and dates. Previous revisions, when appropriate, will be discussed in the ensuing sections.

The methodology in EPRI NP-6695 [EPRI 1989] was used extensively for restart activities of the Kashiwazaki-Kariwa Nuclear Power Plant (KKNPP). The methodology focuses on defining damage states rather than focusing on ground motion comparisons with the Operating Basis Earthquake (OBE) equivalent ground motion or the SSE ground motion, as appropriate. However, the guidance on actions to be taken post-earthquake shaking for evaluations and restart are targeted to OBE ground motion exceedances rather than exceedances of the SSE ground motion, which is important because these higher (than SSE) ground motions are beyond the largest seismic loading considered in the licensing basis. In order to capture the lessons learned from the KKNPP restart experience, the International Atomic Energy Agency [IAEA 2011a], in close collaboration with TEPCO, published a safety report that extends the framework of EPRI NP-6695 to beyond design-basis ground motions. An update to EPRI NP-6695 [EPRI 2012a] was completed in 2012, which took into account some aspects of [IAEA 2011a].

2.1.8 Seismic Performance Objectives for Structures, Systems, and Components

In addition to the legal and other requirements itemized above, two recent advances in the regulation of new NPP design are:

(4) **The requirement that the HCLPF at the plant-level is at least 1.67 times the design ground motion.** The SSE ground motion is defined as peak ground acceleration and associated design ground response spectra. This requirement was described in SECY-93-087 [NRC 1993], “Policy, Technical and Licensing Issues Pertaining to Evolutionary and Advanced Light-Water Reactor Design.” This requirement is intended to ensure that adequate seismic margin is explicitly incorporated into the design. Hence, during the design phase, the plant designer performs a

PRA-based SMA to ensure that appropriate design and qualification criteria are implemented to achieve this margin[3].

- For Certified Designs, this requirement applies to the nuclear island (i.e., the portion of the design that has been licensed under the requirements of 10 CFR 52 and has been designed to the CSDRS). The requirement is that the Certified Design be demonstrated to have a plant-level HCLPF at least 1.67 times the CSDRS.
- For SSCs located in the balance of the NPP that are designed to site specific ground motions (i.e., the ground motion response spectrum or GMRS), the same requirement is imposed (i.e., that a plant-level HCLPF be demonstrated to be at least 1.67 times the GMRS). This requirement may be satisfied by demonstrating that these SSCs individually satisfy the requirement that HCLPFs are at least 1.67 times the GMRS. However, the requirement is for the plant-level HCLPF to be at least 1.67 times the GMRS, not for individual SSCs to be so.

These requirements permitted the implementation of the performance-based approach to specifying the design-basis ground motion described below. As mentioned above, this requirement entails some modification to standard practice to achieve the desired margins. Modifications to standard practice include updates to qualification by testing and treatment of specialized items such as fuel assemblies and steam generator upper supports. For components that are qualified by testing, there is only a 10 percent factor of safety applied to the required response spectra (RRS) and, unless there is significant over testing, or unless the required response spectra are very conservatively defined, the HCLPF for function during the earthquake will not be easily demonstrated to meet the 1.67 times design-basis ground motion criterion. In this respect, Standard ASCE/SEI 43-05 recommends that a load factor of 1.4 be applied to the RRS. This recommendation is not included in the 2004 (latest) version of IEEE-344 or in the current RG 1.100, both of which are deterministic in nature.

(5) **The use of a risk-consistent definition of the design-basis earthquake ground motion (i.e., the SSE ground motion).** The most significant evolution in the design-basis ground motion for an NPP is the use of a risk-informed framework to develop probabilistic ground motion levels to be used in design. This is the natural evolution from the hazard consistent basis of RG 1.165. The risk-informed framework for new reactors is based in part on insights from risk analysis of the currently operating reactors. It is also expected that new plants will have a lower risk profile than the operating reactors.

The first important change in this regard was the development and publication of ASCE/SEI 43-05, “Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities.” ASCE/SEI 43-05 specifies a performance-based approach to defining design-basis ground motions using the results of a probabilistic seismic hazard assessment (PSHA). The approach relies on specified conservatism in the design of SSCs, where the end metric is an acceptable SSC performance defined as *no significant nonlinear behavior*. This document, though not accepted by the NRC in full, provided the framework for the current definition of the site-specific GMRS and, ultimately, the SSE

[3] A seismic probabilistic risk assessment (SPRA) accepted by the NRC is required prior to loading of fuel in a new NPP.

ground motion. The basic performance-based approach is linked to the definition of the SSE ground motion for NPPs through the following elements:

- The risk-informed performance-based approach for developing the GMRS defines a minimum damage state, namely the Frequency of Onset of Significant Inelastic Deformation (FOSID) at a target annual frequency of $1x10^{-5}$/yr. This metric was selected and agreed upon after benchmarking studies were performed using results from the Individual Plant Examination of External Events (IPEEE) program in the United States and the requirement for new plant design to have a plant-level HCLPF equal to or greater than 1.67 times the design-basis earthquake ground motion. Use of the FOSID is beneficial because it is more directly analyzable for an SSC than the core damage frequency (CDF) target.
- The approach utilizes the results of the site-specific PSHA, with site response analyses.
- A scale factor, called a "design factor" and defined in a similar way in both ASCE 43-05 and RG 1.208, is applied to the mean uniform hazard response spectrum to determine the GMRS for the site. The resulting GMRS is "risk-consistent," as described in ASCE/SEI 43-05.
- For verification, the approach was used to analyze 29 NPP sites in the United States using the EPRI PSHA results and the seismic capacities from submittals for IPEEE. The study verified that meeting the FOSID target $1x10^{-5}$/yr leads to seismic CDF values ranging from $5x10^{-6}$ to $1x10^{-6}$.

2.2 Seismic Design Criteria Development, Regulatory Approach, and Framework in Japan

2.2.1 Regulatory Authority up to 2012

Japan has a unique regulatory structure. Up to 2012, the Nuclear and Industrial Safety Agency (NISA) regulated all energy-producing facilities as part of the Ministry of Economy and Trade and Industry (METI), which is responsible for development of nuclear power in Japan. The Nuclear Safety Commission (NSC) was a separate organization appointed by the Prime Minister that checks all regulatory decisions by NISA. Effectively, NISA and METI were required to comply with the nuclear regulation prepared by NSC. However, unlike in the United States, the NSC could not issue licenses or penalize licensees for violations. These functions were the responsibility of METI. While, NISA was the regulatory body, it sat within METI, the organization responsible for promoting nuclear energy. The Japanese Nuclear Energy Safety Organization (JNES) is a technical body that was supporting NISA. JNES does not have regulatory authority, but rather works through NISA. Figure 2-1 shows schematically the relationship between the organizations NSC, NISA, JNES, and the Japanese utilities and their industry-wide organizations that existed at the time of the 2007 and 2011 earthquakes. Figure 2-2 shows the overall flow chart from issuance of the NSC Regulatory Guide (RG) to METI (and through NISA) to the Japanese utilities that existed at the time of the 2007 and 2011 earthquakes.

2.2.2 Regulatory Authority after 2012

Starting in 2012, the structure of the regulatory authority in Japan was changed significantly as a result of the Fukushima Dai-ichi accident caused by the March 11, 2011 Tōhoku[4] earthquake and tsunami. A new regulatory organization named the Nuclear Regulation Authority (NRA) was created by a bill entitled, "Act for Establishment of the Nuclear Regulation Authority" passed on June 20, 2012, by the House of Councilors in Japan (http://www.nsr.go.jp). The stated objective of the change was to increase regulatory independence, integration of regulatory activities, and transparency. The NRA was formed by merging NSC and NISA; and the majority of the organizational changes were implemented in September 2012. At the time of this writing, JNES continues to act as the principal technical supporting organization to the regulator (NRA). However, JNES will be incorporated into the new NRA framework and organization after necessary legal arrangements are made. Figure 2-3 shows schematically the former and current regulation system.

Drafts of new nuclear codes and standards have already been made public. Generally, the new seismic design approaches are consistent with the guidelines published by the NSC[5] on September 19, 2006. Important changes were enacted in the 2006 guidelines, as presented in this report. One of these changes is the recognition of the possibility that earthquake ground motions may occur at a site that exceed the design-basis ground motion. The process of evaluating the effects of beyond design-basis ground motions on the NPP is termed *determining "residual risk"*. The new tsunami guidance is substantially different and more detailed than in the past.

2.2.3 Japan Regulatory Guides and Design Standards

Up to 2012, the NSC of Japan was the source for the primary regulatory documentation for the seismic analysis and design of NPPs. The NSC document "Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities" provides the overall requirements. The "New Guide" [NSC 2006] governs new plant design and safety evaluations of existing plants. Promptly after its issuance, NISA required all utilities to reevaluate the seismic safety of all existing nuclear facilities. This reevaluation is denoted the "Back-Check" and is presented later in this section.

The September 19, 2006, revision to the Japanese regulatory guidance provided the information denoted as Post-2006 [NSC 2006] within this report. Previous versions of the guidance, referred to within the 2006 revision as "Former Guide," were in place for the seismic analysis and design of all NPPs currently operating in Japan. Figure 2-2 shows the overall flow chart from issuance of the NSC RG to METI (and through NISA) to the Japanese utilities as the process stood up to 2012.

The "Former Guide" provided the overall requirements for the development of the comprehensive guidelines named "Technical Guidelines for Aseismic Design of Nuclear Power Plants." These technical guidelines were developed over many years. The first version was published in 1970. Subsequent updates were published in 1984 (adding seismic importance classifications and allowable stresses), 1987, and 1991 (adding definition of damping values for some SSCs and methods of seismic assessment of dynamic components). The 1987 version of these guidelines was translated into English and is published as NUREG/CR-6241, "Technical Guidelines for

[4] Formally called the Tōhoku-chiho Taiheiyo-oki earthquake by the United States Geological Survey, but referred to as the Tōhoku earthquake in this document

[5] NSC was the agency formerly responsible for code and guidance development. It has been dissolved in the new regulatory system.

Aseismic Design of Nuclear Power Plants, Translation of JEAG 4601-1987" [NRC 1994]. It forms the basis for the discussion of U.S. and Japanese practices prior to 2006. Because the current guide [NSC 2006] is available only in Japanese at this time, significant insight and assistance has come through interaction with IAEA, JNES, NISA, and TEPCO staff through the IAEA International Seismic Safety Center (ISSC) and the NRC/JNES Bi-Lateral Research Program.

The major differences between the Former Guide and the NSC 2006 Guide may be summarized as follows:

- The age of active faults is extended from 50,000 years before present to the late Pleistocene age (120,000-130,000 years before present).[6]
- Geomorphological surveys used to identify and characterize tectonic relief are now recommended.
- Both empirical (i.e., ground motion prediction equations) and theoretical (numerical fault modeling) estimation of ground motion are now recommended.
- Unspecified seismic sources near the site are now accounted for directly. In the Former Guide uncertainty was addressed using a generic deterministic earthquake scenario with a 6.5 magnitude at a depth of 10 km.
- Vertical dynamic motions should now be estimated by both a GMPE and a fault model.
- The seismic categorization of SSCs was modified. Former Classes A_s and A were combined into Class S.
- An equivalent static load (as discussed in later sections) continues to be required in JEAC-2008 for seismic design of SSCs.
- The concept of *residual risk* was introduced to account for beyond design-basis ground motions.

A general discussion of pre-2006 detailed guidelines as characterized in NUREG/CR-6241 is presented in this document to provide a historical perspective on the level of detail and types of specifications provided to the nuclear community in Japan and to describe the seismic criteria under which the KKNPP was designed.

The 2006 NSC RG (September 2006) has been complemented by revisions to JEAG 4601. These updated documents, completed and issued by the Japan Electric Association (JEA), are denoted JEAC 4601-2008 (C denotes Code) [JEA 2008a] and JEAG 4601-2008 (G denotes Guidelines) [JEA 2008b]. They were under review by NISA, at the time of the occurrence of the March 2011 Tōhoku earthquake. Currently, the NRA is reviewing the documents, but the schedule for completion and conditions of endorsement are unknown. They are published only in Japanese.[7] Aspects of the implementation that are available through presentations or through interactions with Japanese colleagues are provided when possible.

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[6] The public draft of the new NRA seismic design guideline requires consideration back to 400,000 years (to the middle Pleistocene), if activity is not clear back to 120,000-130,000 years.

[7] At the time of this writing, the NRC is translating portions of the JEAC 2008 and JEAG 2008 as resources allow. However, assuming the code and guidance documents are similar to the 1987 version in terms of their length, it is likely to take a significant amount of time and effort for an English translation to be made.

The Table of Contents for JEAC 4601-2008 (along with the equivalent chapters in earlier JEA guidance) is:

The Table of Contents for JEAG 4601-2008 is:

Back-Check

After issuance of the New Guide, NISA required all utilities to reevaluate the seismic safety of all existing nuclear facilities. The reevaluation includes the following:

- Elements of the reevaluation
  - New geological surveys at the NPP sites
  - Reevaluation of design-basis ground motions (S_s) (see Section 3.2.2)
  - Reevaluation of seismic safety of the NPP facilities
  - Evaluation of ground stability at the NPP sites
  - Consideration of concomitant phenomena such as tsunami, slope stability
- Steps in the reevaluation process
  - Step 1: Determine design-basis ground motion S_s for reevaluation by conducting geological survey
  - Step 2: Reevaluate safety of SSCs to S_s, including at least one unit per site
  - Step 3: Reevaluate all units to the S_s (this step is currently ongoing)
- In 2007, NISA also compiled knowledge and findings from the Niigataken Chūetsu-Oki (NCO) earthquake and directed utilities to consider the information for their sites.
- NISA (or ultimately the new NRA) will review the documentation provided by the licensees once submitted. JNES is expected to support this effort by cross-checking typical plant results.
- In response to the reevaluation, utilities are implementing seismic retrofits and upgrades as required; although at the time of the Tōhoku earthquake the upgrades were voluntary.

Residual Risk

The NSC RG of September 19, 2006, recognizes the possibility of beyond design-basis earthquake ground motions occurring, denoting it "residual risk." This residual risk is required to be considered. The guidelines within JEAG 4601-1987 [see translation in NUREG/CR-6241] did not specifically address the possibility of larger than S_2 earthquake ground motions (as described in Section 3.2.1) occurring. Reevaluations of existing plants are ongoing and it is understood that larger earthquake ground motions than the S_s may need to be considered.

For Japanese new designs, seismic margin and seismic probabilistic safety assessment (PSA)[8] approaches are being implemented to demonstrate margin, at least for NPPs being offered to countries other than Japan. In the past, the generally high levels of performance of Japan's NPPs that have been subjected to ground motions larger than design earthquake motions have indicated that significant margin against seismic shaking has been introduced into the design process[9].

The two principle sources used in this report for review of the Japanese approaches are:

Pre-2006 NUREG/CR-6241, "Technical Guidelines for Aseismic Design of Nuclear Power Plants, Translation of JEAG 4601-1987"

Post-2006 Nuclear Safety Commission, "Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities," September 19, 2006, JEAC 4601-2008 and JEAG 4601-2008

This formal material was supplemented by presentations made by Japanese experts at many meetings. In addition, numerous Japanese codes are referenced in JEAG 4601-1987. The Building Standard Law of Japan is translated into English (http://ww.bcj.or.jp/en/index.html), but was not studied in detail for this effort. Translation of the seismic PSA standard written by the Atomic Energy Society of Japan is in progress through the NRC-JNES bi-lateral program. An overview of the content of the Japanese Society of Civil Engineers tsunami evaluation standard (2002) that is available only in Japanese has been published as an Annex of the IAEA Safety Guide on hydrological and meteorological hazards [IAEA 2011b]. Other Japanese codes are unavailable in English and, thus, not utilized for this effort.

It is clear that these two documents (pre- and post-2006) belong to two different tiers of hierarchy of regulatory guidance and so a direct comparison is difficult. The preparation of the new revision of JEAG 4601 was completed in 2008 and the Tables of Contents were presented above. However, a translation into English is not available.

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[8] Seismic PSA is similar to seismic PRA in the US

[9] At the time of this writing, it is unclear how much damage from the 2011 Tōhoku earthquake was due to shaking versus the tsunami and other complicating factors.

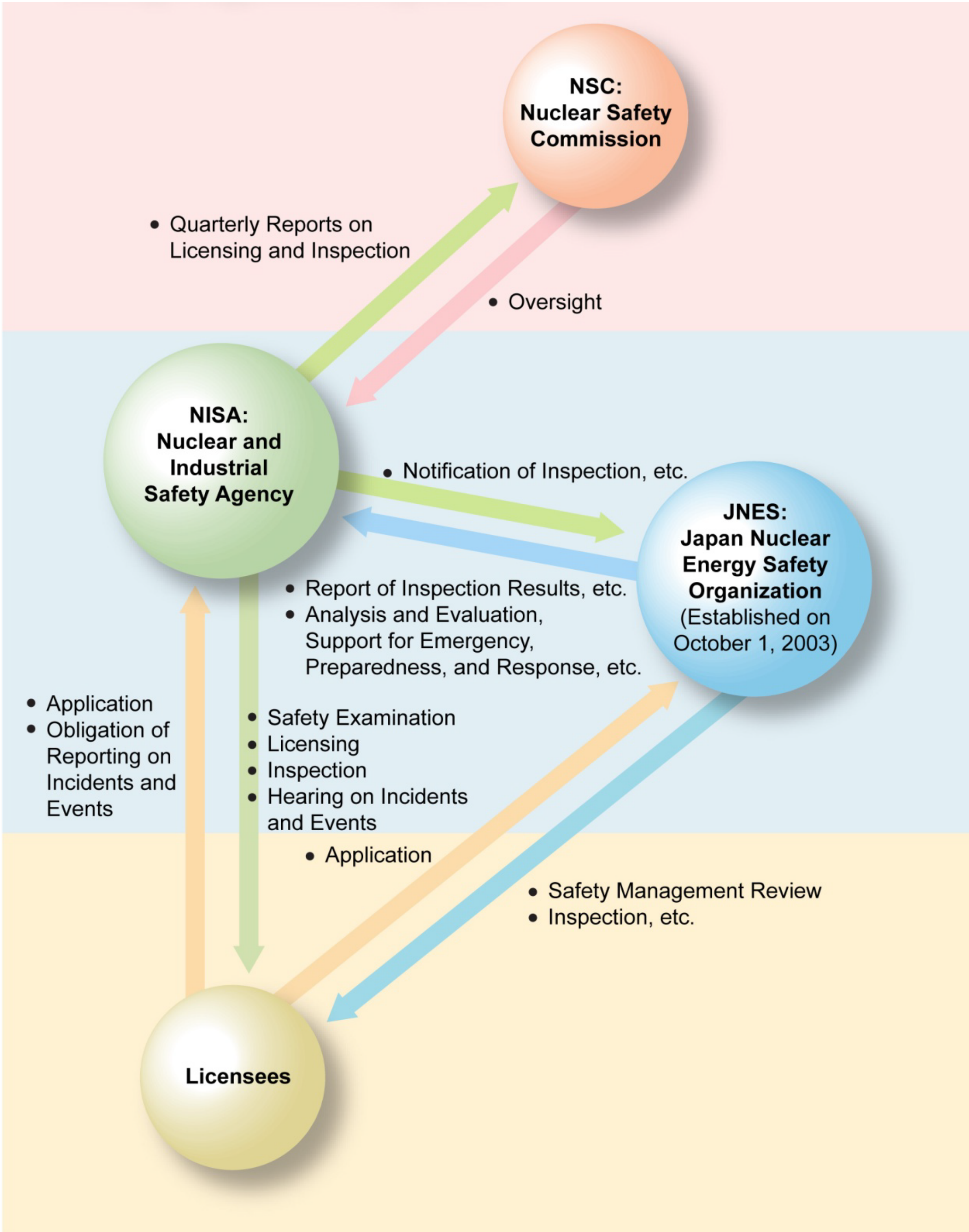


Figure 2-1 Overview of the nuclear safety regulation system in Japan at the time of the 2007 Niigataken Chūetsu-Oki (NCO) and 2011 Tōhoku earthquakes

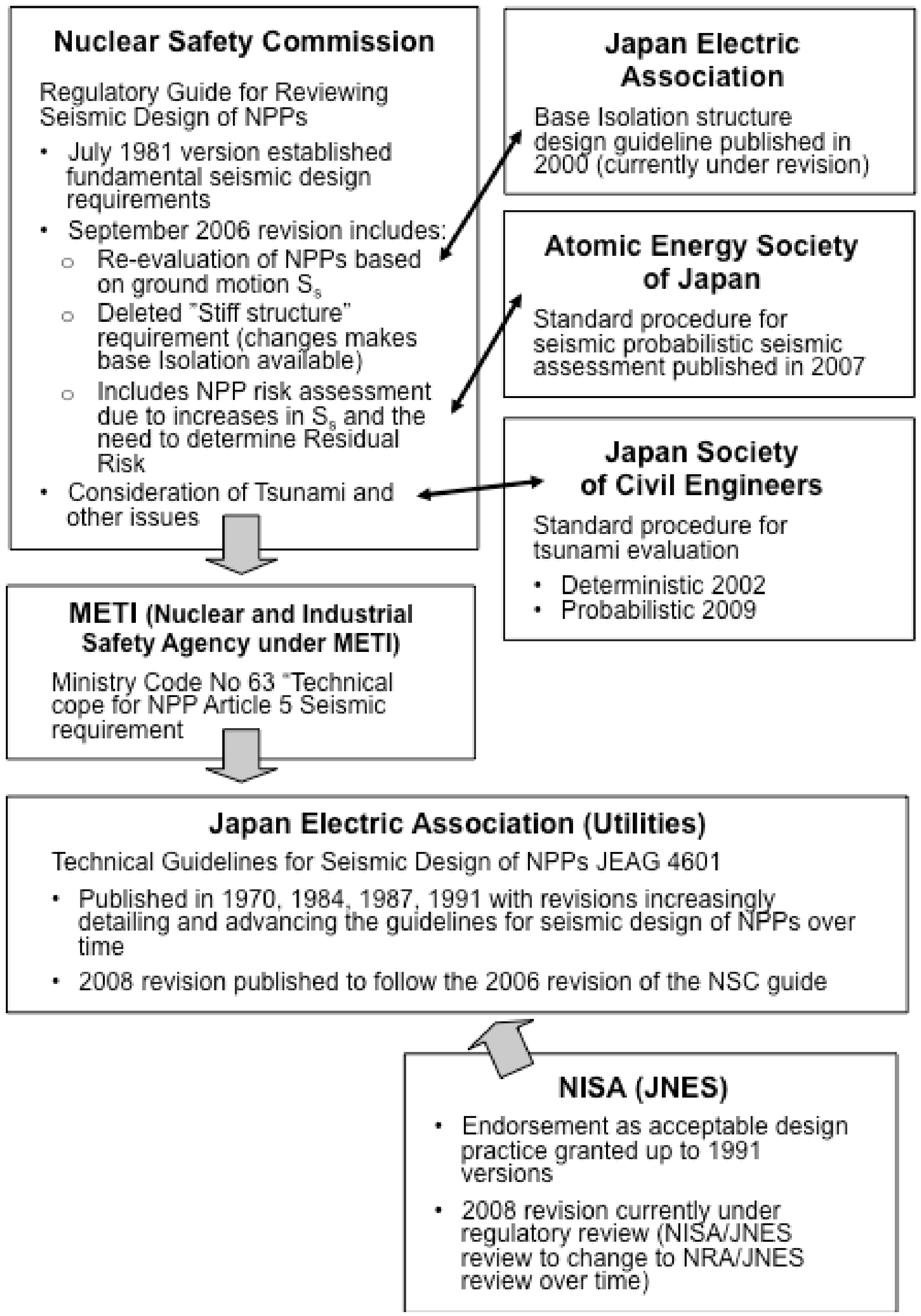
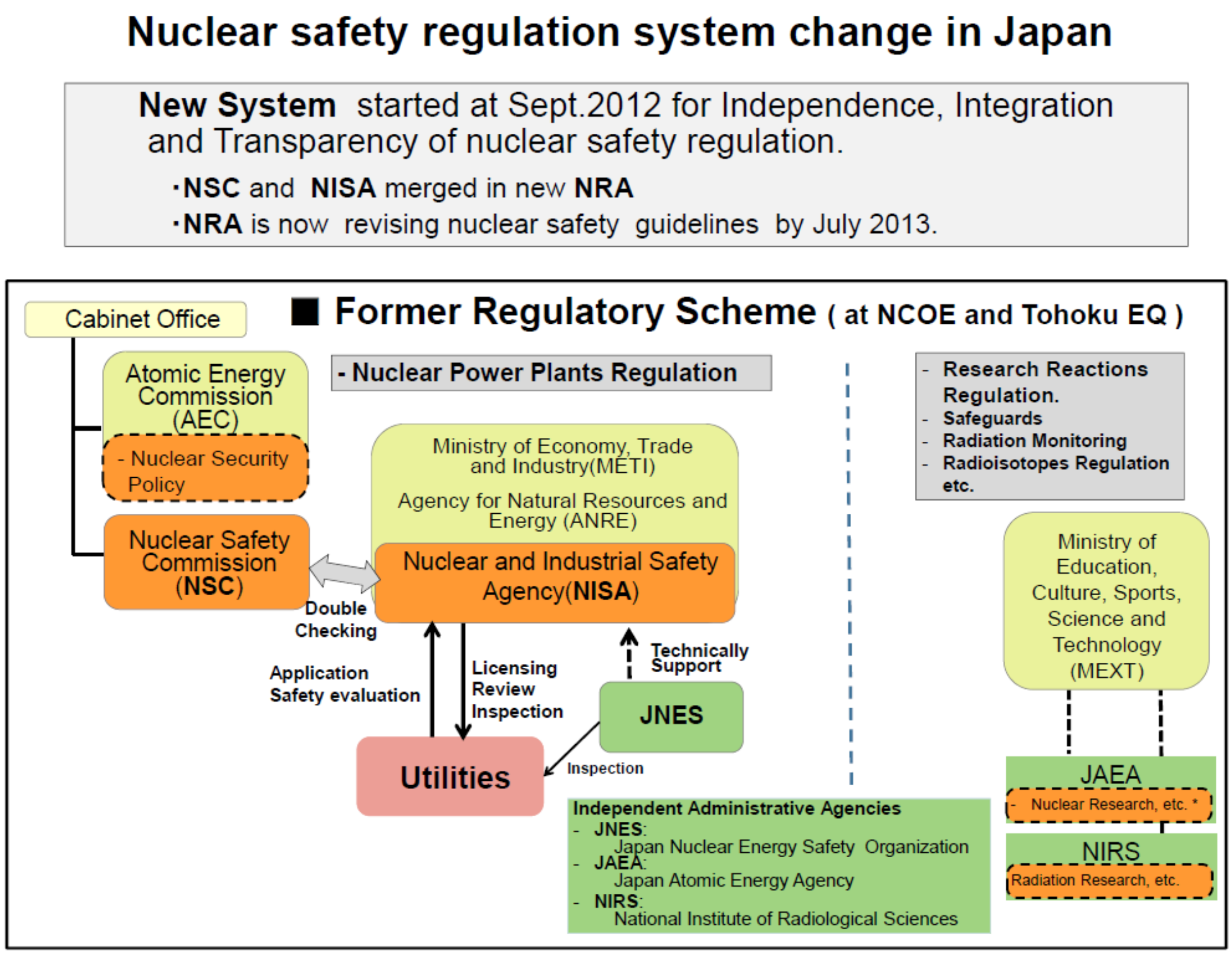


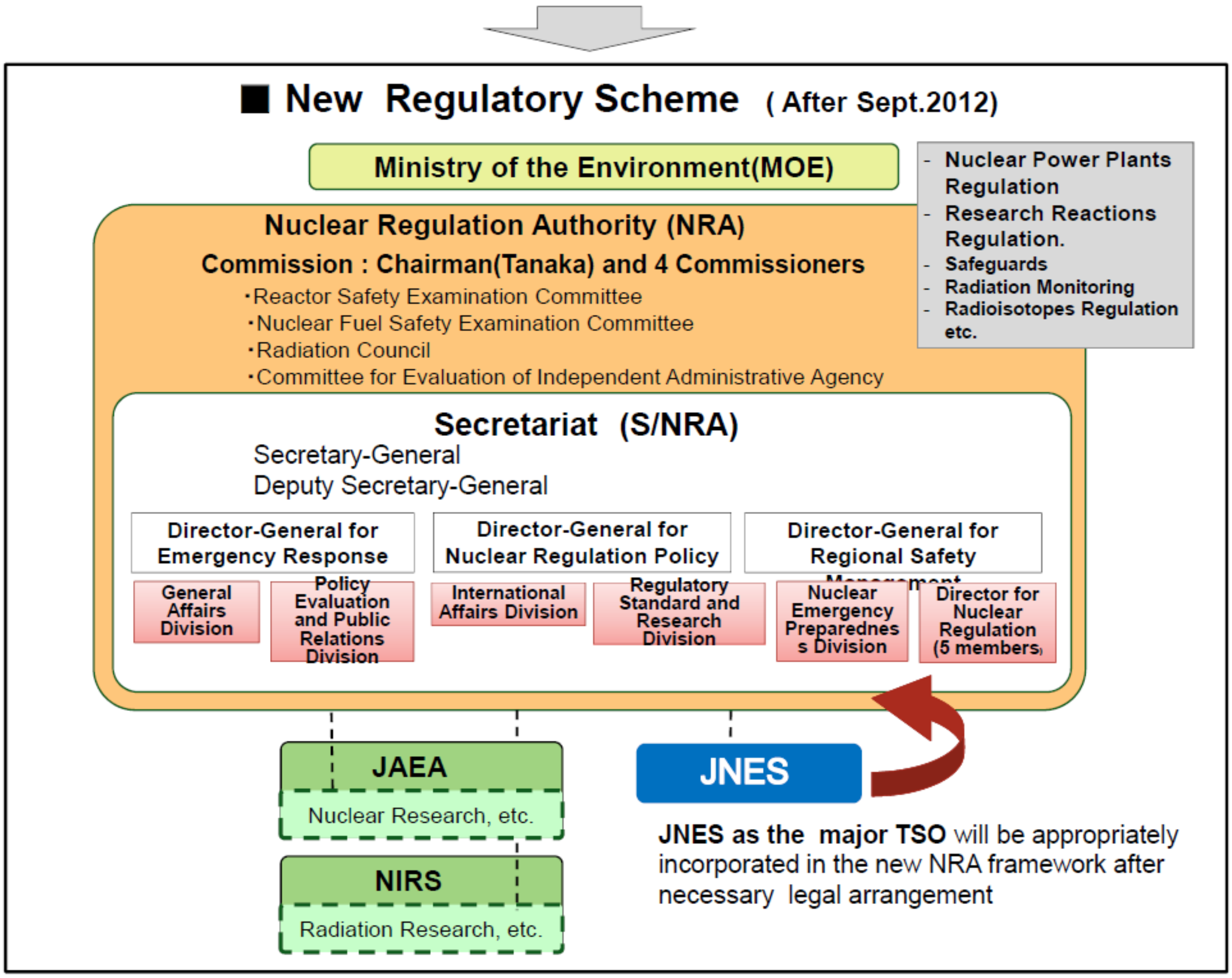
Figure 2-2 Flow chart of the safety regulation system in Japan at the time of the 2007 Niigataken Chūetsu-Oki (NCO) and 2011 Tōhoku earthquakes



(a)

Figure 2-3 Changes to nuclear regulatory structure in Japan as of September 2012 (based on figures created for the NRC by JNES)

(a) Former Japan Regulatory Structure (pre-September 2012)



(b)

Figure 2-3 **Changes to nuclear regulatory structure in Japan as of September 2012 (based on figures created for the NRC by JNES)**
(b) New Japan Regulatory Structure (post-September 2012)

3 SEISMIC DESIGN CRITERIA AND DEVELOPMENT OF THE DESIGN-BASIS GROUND MOTION IN THE UNITED STATES (U.S.) AND JAPAN

3.1 Definition of the Design-Basis Ground Motion in the United States

3.1.1 Historical Perspective on Ground Motion Criteria for the U.S.

Prior to 1973, the Design-Basis Earthquake (DBE) ground motion for nuclear power plants (NPPs) was based on a handful of recorded acceleration seismograms that had been recorded at that time (e.g., the El Centro and Golden Gate recordings) and standard ground motion response spectra developed from a series of recorded motions that were smoothed and enveloped (e.g., Housner average ground response spectra [AEC 1963]). These definitions were accompanied by required or recommended analysis techniques and parameter values to generate design quantities (e.g., forces and moments). Generally, these analysis techniques were relatively simple, but were considered to be conservative, and were consistent with the state-of-the-art of earthquake engineering at the time[1]. Important parameter values such as damping were very conservatively specified. Often, the combination of these design-basis ground motions, analysis techniques, and parameter values applied as an integrated process led to designs that have been shown to have high levels of safety even by more recent procedures. Plants where adequate safety cannot be shown have been decommissioned.

Geologic and seismic regulatory practice in the *Code of Federal Regulations* (CFR) evolved during this early period culminating in the issuance of Appendix A to 10 CFR Part 100, “Seismic and Geologic Siting Criteria for Nuclear Power Plants,” issued in 1973. Further, in 1973 the U.S. Nuclear Regulatory Commission (NRC) issued Regulatory Guide (RG) 1.60, “Design Response Spectra for Seismic Design of Nuclear Power Plants,” which defined standard ground response spectra shapes for design. The design-basis ground motion for NPP units at 30 sites in the United States are defined by RG 1.60 response spectra anchored to a site-specific peak ground acceleration (PGA) determined by deterministic (scenario-based) hazard analyses.

Although the DBE ground motion is now defined using modern probabilistic techniques (and is called the safe shutdown earthquake (SSE) ground motion), the RG 1.60 response spectrum still plays a role in the definition of the design-basis ground motion for low seismicity sites as a result of the minimum seismic design requirements (i.e., the 10 CFR Part 50, Appendix S, minimum design requirement of PGA of 0.1g anchoring appropriate response spectral shape at foundation level in the free-field). According to Standard Review Plan (SRP) Section 3.7.1, one way of satisfying the minimum seismic input requirement of 10 CFR Part 50, “Domestic Licensing of Production and Utilization Facilities,” Appendix S, “Earthquake Engineering Criteria for Nuclear Power Plants,” is through use of the 0.1g required minimum PGA anchored to a RG 1.60 ground motion response spectrum. In some cases, RG 1.60 at foundation level in the free-field can lead to very conservative definitions of the seismic hazard on the free surface top-of-grade for some spectral frequencies. Additionally, some of the Certified Designs are designed to a Certified

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[1] More recent work by NRC staff has indicated that modern seismic hazard assessment techniques produce results that routinely exceed the original design-basis ground motions of NPPs [NRC 2010c]. The exceedance of the SSE ground motion at the North Anna NPP during the August 2011 Mineral, Virginia, Earthquake supports the updated assessment results.

Seismic Design Response Spectrum (CSDRS) based on either the RG 1.60 response spectrum or the RG 1.60 response spectrum augmented in the high frequency range.

NPPs licensed prior to January 1997 comply with the seismic siting requirements of 10 CFR Part 100, Appendix A. Those licensed after January 1997 must comply with the geologic and seismic siting criteria of 10 CFR 100.23, "Geologic and Seismic Siting Criteria," and the seismic design requirements of 10 CFR Part 50, Appendix S.

For the purpose of this document, the following Table represents the key CFR requirements, RGs, and SRP sections that were considered in the comparison of approaches for design-basis ground motion development:

Table 3-1 List of key CFR requirements, RGs, and SRP sections considered in comparison approaches

10 CFR Part 100	"Reactor Site Criteria," Appendix A, "Seismic and Geologic Siting Criteria for Nuclear Power Plants"
10 CFR 100.23	"Geologic and Seismic Siting Criteria"
10 CFR Part 50	"Domestic Licensing of Production and Utilization Facilities," Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants"
10 CFR Part 52	"Licenses, Certifications, and Approvals for Nuclear Power Plants"
RG 1.60	"Design Response Spectra for Seismic Design of Nuclear Power Plants," Revision 1, 1973
RG 1.165	"Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion," March 1997 (withdrawn April 30, 2010)
RG 1.208	"A Performance Based Approach to Define the Site-Specific Earthquake Ground Motion," March 2007
SRP 2.5.2	"Vibratory Ground Motion," Revision 4, March 2007
SRP 3.7.1	"Seismic Design Parameters," Revision 3, March 2007

For purposes of this discussion, three general time frames are relevant: about 1973 to 1996, 1997 to 2007, and beyond 2007.

Additionally, at the time of this writing, the NRC has issued a Request for Information letter pursuant to 10 CFR 50.54(f) to licensees of all operating reactors in the United States [NRC 2012c]. Responding to the NRC letter request requires that seismic hazard and risk reevaluation activities be undertaken by the licensees. The reevaluation to be performed is based on Recommendation 2.1 in the NRC's post-Fukushima Near Term Task Force Report [NRC 2011]. As described in Enclosure 1 of that document, the seismic hazard reevaluation should use a probabilistic approach as defined in RG 1.208, NUREG/CR–6372, "Recommendations for Probabilistic Seismic Hazard Analysis: Guidance on Uncertainty and Use of Experts," [NRC 1997b] and NUREG-2117, Revision 1 "Practical Implementation Guidelines for SSHAC Level 3 and 4 Hazard Studies," [NRC 2012b]. Plants in the central and eastern United States may use a recently developed seismic source characterization model published in January 2012 as NUREG-2115, "Central and Eastern United States Seismic Source Characterization for Nuclear

Facilities" [NRC 2012a] along with previously published ground motion prediction equations (GMPEs)[2]. Plants not in the NUREG-2115 study area (i.e., the four western sites) will perform site-specific Senior Seismic Hazard Analysis Committee (SSHAC) level 3 hazard studies. Additional guidance for response to the 10 CRF 50.54(f) letter has been developed. Electric Power Research Institute (EPRI) report 1025287 [EPRI 2012b], which was endorsed by the NRC, provides guidance on hazard assessment and seismic probabilistic risk assessment (SPRA). The NRC also published new guidance on conducting seismic margin assessments "Interim Staff Guidance on Performing a Seismic Margin Assessment in Response to the March 2012 Request for Information Letter" [NRC 2012d].

3.1.2 U.S. Criteria from 1973 to 1996

Throughout the period from 1973 to 1996, both an Operating Basis Earthquake (OBE) ground motion and an SSE ground motion were required to be considered in the seismic design, although the terminology differed in some cases. The horizontal and vertical ground response spectra shapes were defined by RG 1.60 anchored to PGA values derived from deterministic seismic hazard assessments. In the central and eastern United States (CEUS), the largest horizontal SSE PGA is 0.25g for the Seabrook Station in New Hampshire. In the western United States (WUS), the largest horizontal SSE PGA is 0.67g for the San Onofre Nuclear Generating Station in California. The Diablo Canyon Power Plant (DCPP) site in California was originally designed for an SSE PGA of 0.4g, but currently also has a third licensing basis ground motion (called the Hosgri Earthquake ground motion) with PGA of 0.75g, for which the DCPP has been evaluated and modified. This is similar to the updated S_s value for the NPPs in Japan in the sense that it is a third licensing basis ground motion that is higher than the SSE ground motion, and for which the NPP has been reevaluated. For more information on the DCPP licensing basis, see NRC Research Information Letter 12-01 "Confirmatory Analysis of Seismic Hazard at the Diablo Canyon Power Plant from the Shoreline Fault Zone" [NRC 2012e].

From 1973 onward, three components of ground motion (2 horizontal and 1 vertical) were required to be considered in the seismic design. The OBE ground motion was most typically taken to be one-half of the SSE. In a number of situations, the OBE ground motion dictated the design for some Seismic Category-I (SC-I) structures, systems, and components (SSCs), because the permissible material damping values and allowable stress design limits are often significantly lower than those for the SSE ground motion.

Within this time frame, extensive site-specific assessments were also performed using the newer probabilistic seismic hazard analysis (PSHA) techniques. For CEUS sites, the NRC (through Lawrence Livermore National Laboratory (LLNL)) and EPRI were both conducting parallel PSHAs. The end results provided both data and the technical bases for the development of the approach later specified in RG 1.165 for defining SSE ground motion. Further, the PSHA results were used by licensees as a basic input to their Individual Plant Examination of External Events (IPEEE) program. The PSHA results provided one criterion for binning plants as a function of the seismic hazard, including consideration of the relationship between the seismic hazard and the SSE ground motion to which the plant had been designed. The PSHA results also provided essential input of the seismic hazard (seismic hazard curves and uniform hazard spectra) for those plants that addressed IPEEE using the SPRA approach.

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[2] Ground motion prediction equations are also known as attenuation relationships.

As a result of the differences in results of the LLNL and EPRI studies, the NRC and Department of Energy (DOE) sponsored work by the SSHAC to review the two studies. In 1997, the NRC issued NUREG/CR-6372, “Recommendations for Probabilistic Seismic Hazard Analysis: Guidance on Uncertainty and the Use of Experts.” The document was the culmination of 4 years of deliberations by the SSHAC regarding the manner in which the uncertainties in PSHA should be addressed using expert judgment. The document describes a formal process for structuring and conducting expert assessments that has come to be known as a “SSHAC process,” and the recommendations made in the report are referred to as the “SSHAC guidelines.” To account for different project needs and projects undertaken in different regulatory contexts, the SSHAC report describes four “Study Levels” that define the processes and complexity of the recommended project activities. SSHAC Study Levels 3 and 4 are the most complex and involve the greatest amount of effort. These are the only levels allowed by the NRC for new studies; although SSHAC Level 2 studies may be used to update regional SSHAC 3 or 4 studies if the underlying regional model is still generally valid. Regardless of the study level, the development of a model that represents the center, body, and range of the technically defensible interpretations is the key objective.

The NRC recently published NUREG-2117, “Practical Implementation Guidelines for SSHAC Level 3 and 4 Hazard Studies.” This new NUREG was developed based on lessons and insights gained over the previous 15 years applying the SSHAC guidelines and provides in depth recommendations for undertaking SSHAC-based hazard assessment studies, particularly using the Level 3 approach. This new NUREG report also provides information on how and when to update a SSHAC-based study. The SSHAC guidelines are the state-of-practice for performing a PSHA for a nuclear site in the United States and are referenced in RG 1.208.

3.1.3 U.S. Criteria from 1997 to 2007

With the perceived maturity of the PSHA methodologies, a risk-based approach for the definition of the design-basis ground motion evolved. Regulations in 10 CFR 100.23 require that uncertainty inherent in estimates of the SSE ground motion be assessed through an appropriate analysis, such as a PSHA or suitable sensitivity studies.

RG 1.165 served as an interim measure during this time frame and bridged the approaches applied prior to 1997 to the approach in RG 1.208. RG 1.165 defined an acceptable approach to the definition of the SSE ground motion incorporating uncertainties. The basic principle was that new plants should be designed to earthquake levels (in terms of annual probability of exceedance) that are more conservative than the median design ground motions for a set of currently operating NPPs in the United States. The set of plants defining this basis was selected by the NRC and was comprised of twenty-eight sites at which newer nuclear power units were operating. For the selected sites, PSHAs were to be performed using a methodology that was the state-of-the-art at the time. Two examples of such PSHA approaches were the LLNL and EPRI methodologies discussed above, both of which used advanced techniques and state-of-the-art models. Application of the LLNL methodology for the 28 sites defined the probabilities of exceedance of the SSE ground motions. The median value of the annual probability of exceedance of the SSE ground motions for the twenty-eight sites was evaluated and judged to be approximately 1×10^{-5}. This value was then defined as the “reference probability” and had the attribute that 50 percent of the sites had SSE ground motions with annual probability of exceedance above this value and 50 percent had SSE ground motions that fell below. Therefore, the definition of SSE ground motion (spectral shape and peak ground motion parameters) for new units on an existing site or a new site was targeted at a median probability of exceedance of 1×10^{-5}. RG 1.165 provided significant details on the acceptable approach to determining the intermediate steps of the hazard assessment, as well as the end result. Finally, the end result was to be compatible with the guidance of SRP Section 2.5.2. In some cases, to do so, the resulting SSE ground motion was selected to be a broad-banded spectrum that enveloped the site-specific ground motion. The response spectrum in RG 1.60 was considered to have an acceptable broad-banded spectral shape.

In this same time frame, 10 CFR Part 50, Appendix S, was issued, which specifies a minimum design requirement of PGA of 0.1g anchoring an appropriate response spectral shape at foundation level in the free-field. This requirement is to ensure that there is adequate frequency content in the DBE ground motion.

RG 1.165 was withdrawn on April 30, 2010. It has been replaced by RG 1.208. According to the NRC, "Regulatory Guide 1.165 was withdrawn and replaced with the improved guidance in RG 1.208 which incorporates new developments in ground motion estimation models; updated models for earthquake sources; methods for determining site response; and new methods for defining a site-specific, performance-based ground motion response spectrum (GMRS)." (*Federal Register*/Vol. 75, No. 83/Friday, April 30, 2010, pp. 22868-22869) The criteria in 10 CFR Part 50, Appendix S, still apply. RG 1.208 noted that the methodology of RG 1.165 is hazard-based rather than risk-based; a risk-based approach is needed for performance-based design criteria.

3.1.4 U.S. Criteria Post-2007

Several factors contributed to the evolution of the methodology to define the design-basis ground motion over the last few years: the Commission subsidiary benchmark goal for accident prevention that no single cause has an annual core damage frequency (CDF) of greater than $1x10^{-5}$; the development and publication of standard ASCE/SEI 43-05, which specifies a performance-based approach to defining design-basis ground motions for critical facilities (the performance criteria used in standard ASCE/SEI 43-05 is damage limitations of SSCs); and the requirements for newly designed NPPs to demonstrate seismic margin to earthquake ground motion levels 1.67 times the design-basis earthquake ground motion. All of these elements combined led to the performance-based approach specified in RG 1.208 and reiterated in SRP Section 3.7.1.

SRP Section 3.7.1 *Seismic Design Parameters*, Revision 3, updated in March 2007, deals principally with the definition of the ground motion for the various cases of interest. Reference is made to SRP Section 2.5.2 for ground motion definition, 10 CFR 50, Appendix S, for the minimum design requirement of 0.1g anchoring an appropriate response spectral shape at foundation-level in the free-field, and RGs 1.60, 1.61, 1.165, and 1.208.

Generally, revisions to SRP Section 3.7.1 were necessary to address the new definitions related to the design-basis ground motion due to the performance-based approach detailed in RG 1.208 and the situations that arise for Certified Standard Plant Design and Non-Standard Plant Design. These include new approaches to demonstrate satisfaction of the minimum design requirement for foundation level motions. Other issues such as requirements for ground motion acceleration time series[3] use for structural analysis (e.g., three spatial components, power spectral density criteria, single sets versus multiple sets), site conditions, damping, and other seismic elements remained consistent with Revision 2 of SRP Section 3.7.1.

[3] Acceleration time series are also frequently called "time histories"

The appropriate licensing framework to be used for the assessment of hazard for new reactors depends on whether or not the NPP to be constructed is a Certified Design. **Table 3-2** defines important terms related to ground motion definitions that are provided in RG 1.208. Figures 2 and 3 of SRP Section 3.7.1 (provided as Figures 3-1 and 3-2 of this report) show flow charts of the decision trees for development of the design-basis ground motions for the cases where a Certified Design is and is not used. The frameworks are characterized by the following set of information.

(1) Certified Standard Plant Design

- An original goal of the Certified Design concept was the development of standard designs suitable for 80 percent of the sites in the CEUS, although the Certified Seismic Design Response Spectra (CSDRS) is chosen by the NPP vendor.
- The CSDRS defines the design ground motion for the nuclear island of the Certified Design. Attributes of the CSDRS include the following:
  - A broad-banded response spectral shape is used (in some cases a RG 1.60 spectral shape augmented in the high frequency range (greater than 10 Hz) to accommodate PSHA results for rock sites has been used).
  - The CSDRS is site-independent.
  - For many designs, the control point at which the CSDRS is specified is at foundation level in the free-field as an outcrop independent of the site condition.
- GMRS (Ground Motion Response Spectra) are defined on a site-specific basis using the guidelines of RG 1.208. Attributes of the GMRS include the following:
  - A single response spectrum shape is derived from the uniform hazard response spectrum (UHRS) for annual probabilities of exceedance of $1x10^{-4}$ and $1x10^{-5}$. A design factor is derived from the amplitude ratio between the $1x10^{-4}$ and $1x10^{-5}$ UHRS. The design factor is applied to the $1x10^{-4}$ UHRS to obtain the GMRS. The GMRS incorporates the full range of frequencies derived from the PSHA.
  - The site-specific GMRS is compared to the CSDRS to assess suitability of the Certified Standard Plant Design for the site. To make this comparison, the CSDRS and the GMRS are required to be at the same location and in the same form (outcrop or in-column motions).
  - The definition of GMRS was developed based on the performance-based approach.
  - The vertical ground response spectrum is defined in terms of horizontal ground response spectra.
- The GMRS is applicable to the seismic design of site-specific SSCs, including the balance-of-plant that encompasses the areas outside the Certified Design. The GMRS can be used for the assessment of geologic foundation materials, for assessment of secondary site effects (e.g., liquefaction and landslide susceptibility assessment).
- Figure 1 of SRP Section 3.7.1 (summarized in Figure 3-1 here) provides a flow chart of the process of evaluating the CSDRS, GMRS, and the minimum

foundation input response spectrum requirement to arrive at the design-basis ground motion.

(2) Non-Standard Plant Design

- The GMRS heavily influence the process to determine the SSE ground motion for all SSCs.
  - The approach in RG 1.208 is to be used to determine the GMRS[4]
  - PSHA results (seismic hazard curves and UHRS) for low frequency events (1 and 2.5 Hz) and high frequency events (5 and 10 Hz) may be enveloped or treated separately in the design process.
  - Standard broad-banded response spectra such as the RG 1.60 spectrum may be tied to a zero period acceleration of 0.1g for the purposes of assuring that the minimum design requirement at the foundation level is met (the resulting ground motion on the surface of the soil resulting from this foundation level motion may be overly conservative for some frequencies)
- Figure 3 of SRP Section 3.7.1 (reproduced here as Figure 3-2) provides a flow chart of the process of evaluating the GMRS, enveloping, and the minimum requirement to arrive at the design basis

An important assumption in RG 1.208 is the explicit acknowledgement of the existence of uncertainty in many phases of the complete seismic design criteria, including not only the seismic hazard, but also the performance of the SSCs when subjected to earthquake motions. Further, it requires that uncertainty be taken into account in the process.

There are four key elements in the approach defined in RG 1.208:

(1) Site and regional specific geological, geophysical, and geotechnical investigations are to be performed. This element entails developing an up to-date, site-specific, earth science database to support site characterization and PSHA activities. Emphasis is given to using existing data as the starting point to updates as necessary.

(2) PSHA must be performed to determine the GMRS. The four important steps to the PSHA are:

   1. Identification of seismic sources and development of models (or alternate models) describing their key characteristics (e.g., fault type or mechanism, location or area source boundaries, maximum magnitude) based on regional and site investigations.

   2. Development of earthquake recurrence relationships for each seismic source. This involves identifying appropriate relationships and including alternatives with weighting factors.

   3. Determination or development of appropriate GMPEs[5] for each seismic source. This includes considering alternatives with weighting factors.

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[4] In a few cases, RG 1.165 is being used to determine the GMRS because an application in process was submitted using RG 1.165 when the guide was still applicable.

[5] GMPEs were formerly called attenuation relationships

4. Performing seismic hazard calculation.

An appropriate PSHA model requires the development of both a source characterization model and a ground motion characterization model. Both models are composed of sets of alternate models, methods, and interpretations. A logic tree approach is typically used to weight and communicate credible alternative seismic source models and parameters (in order to account for epistemic uncertainties). Aleatory variability is accounted for through integration over probability distributions representing uncertainties in the parameters of interest. SSHAC Level 3 or 4 studies need to be completed for both the seismic source and ground motion characterization.

The end products of PSHA include seismic hazard curves that provide annual probability of exceedance of ground motion parameters, UHRS, design response spectra, and information on the earthquake scenarios (magnitude and distance) that contribute most to hazard as provided by the deaggregation. It should be noted that the objective of PSHA is to develop an accurate assessment of seismic hazard that also captures the true uncertainty range. The concept of “conservatism” doesn’t make sense in relation to PSHA because nearly every ground motion level is possible with some probability. This is in contrast to deterministic analyses that simply assume that a higher value is “more conservative” without having an understanding of the return period of any particular ground motion.

This information must be provided over a range of relevant outputs that include:

- Discrete frequencies (1, 2.5, 5, 10, and 25 Hz.)
- Annual probability of exceedance (10^{-4}, 10^{-5}, and 10^{-6})
- Confidence levels (15 percent, 50 percent, 85 percent, and mean)

Although the PSHA methodology employed is the same, differences exist between the way that seismic sources behave in the CEUS and WUS that are important to development of the seismic source characterization models used in the PSHA. In the CEUS, there are limited areas where faults are identifiable (e.g., the Meers and Charleston faults) and background and area sources tend to control the hazard results (see NUREG-2115 for an extensive discussion on this topic). This is in contrast to the WUS, where fault systems are often important and dominate the seismic hazard at a site. In both cases, seismic hazard curves and UHRS are usually calculated on a real or hypothetical rock outcrop and site-specific soil amplification analyses are performed to obtain motion for design (e.g., GMRS). In both the WUS and the CEUS, the definition of rock for the GMPEs used in the PSHA should determine the location of the rock horizon in the geologic profile if site-response is to be performed. In the CEUS, GMPEs generally define hard rock as geologic materials with $V_s \geq 9{,}200$ fps; while in the WUS, the V_s is lower.

(3) Site response analysis performed as part of the seismic hazard analysis generally uses one-dimensional wave propagation approaches to generate motion at locations of interest within the site profile based on the PSHA seismic hazard results on rock and taking into account nonlinear behavior of the soil, uncertainties in soil profile stratigraphy, and uncertainties in material properties. Locations of interest in the profile are top-of-grade and at foundation level of Seismic Category I (SC-I) structures. NUREG/CR-6728, “Technical Basis for Revision of Regulatory Guidance on Design Ground Motions,” [NRC 2001b] provides acceptable methods to incorporate local site response into the seismic hazard assessment. Several approaches to incorporating site response are described in NUREG/CR-6728 and approaches 2a and 3 are the most common that have been used recently for assessment of new and operating reactor sites.

(4) Risk consistent definition of the design-basis ground motion (i.e., the GMRS) is used (see Section 2.1).

Table 3-2 Key definitions related to design-basis ground motions (paraphrased and expanded from NRC Regulatory Guide 1.208)

Certified Seismic Design Response Spectra (CSDRS) – Site-independent seismic design response spectra that have been approved under Subpart B, "Standard Design Certifications," of Title 10, Part 52, "Early Site Permits: Standard Design Certifications (DCs); and Combined Licenses (COLs) for Nuclear Power Plants," of the *Code of Federal Regulations* (10 CFR Part 52) as the seismic design response spectra for an approved certified standard design nuclear power plant.
Ground Motion Response Spectra (GMRS) – Site-specific ground motion response spectra characterized by horizontal and vertical response spectra determined as free-field motions on the ground surface or as free-field outcrop motions on the uppermost in-situ competent material using performance-based procedures in accordance with RG 1.208.
Foundation Input Response Spectra (FIRS) – The Performance-based site-specific seismic ground motion spectra at the foundation levels in the free-field are referred to as the FIRS and are derived as free-field outcrop spectra. The FIRS is the starting point for conducting a soil-structure interaction (SSI) analysis and making a one-to-one comparison of the seismic design capacity of the standard design and the site-specific seismic demand for the site. The FIRS for the vertical direction is obtained with the vertical to horizontal (V/H) ratios appropriate for the site.
Safe Shutdown Earthquake (SSE) - The Safe Shutdown Earthquake (SSE) ground motion for a site is: (i) the CSDRS for the Certified Design portion of the plant; and (ii) the GMRS (supplemented if necessary to satisfy the minimum requirement of 10 CFR Part 50, Appendix S) for which site-specific designs are performed, e.g., liquefaction, slope stability.
Operating Basis Earthquake (OBE) - The Operating Basis Earthquake (OBE) for a site is: (i) for the Certified Design portion of the plant, the OBE ground motion is one-third of the CSDRS; (ii) for the safety-related non-certified design portion of the plant, the OBE ground motion is one-third of the ground motion response spectra (supplemented if necessary to satisfy the minimum requirement of 10 CFR Part 50, Appendix S). For determining OBE exceedance (RG 1.166), OBE is the lowest of (i) and (ii).

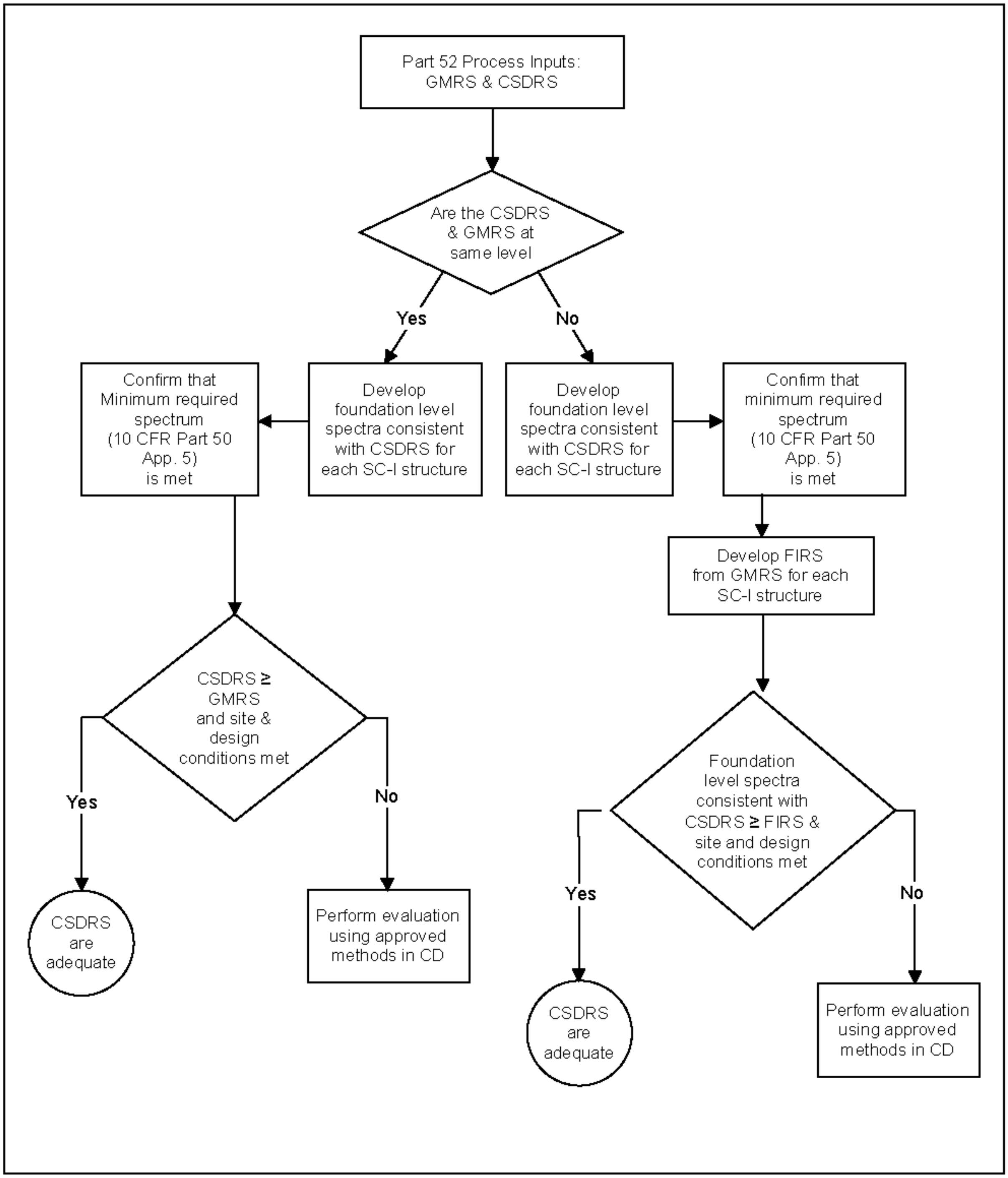

Figure 3-1 Process to define the design-basis ground motion for sites where a Certified Design is not used (Figure 3 of Appendix D of SRP Section 3.7.1)

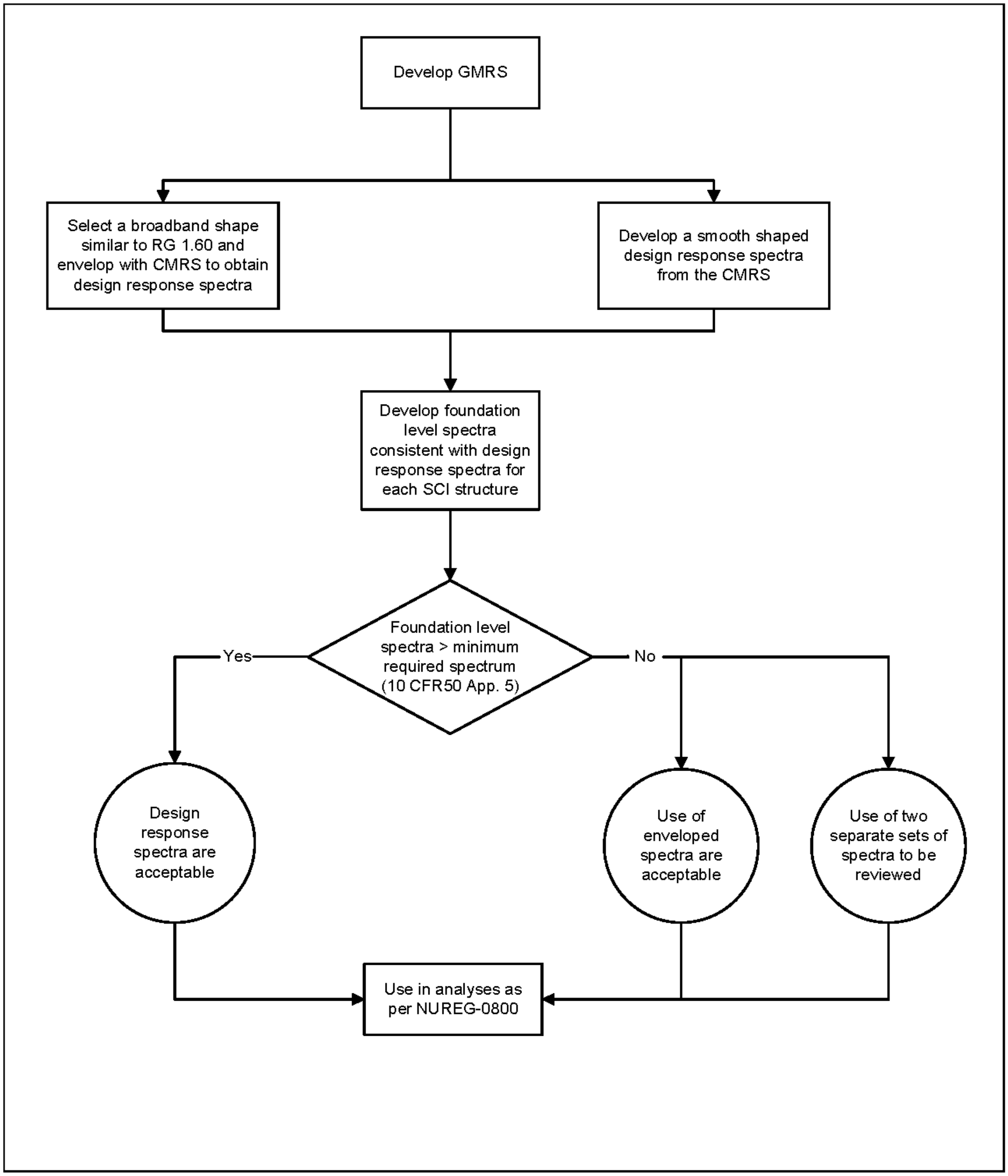

Figure 3-2 Process to define the design-basis ground motion for sites where a Certified Design is not used (Figure 3 of Appendix D of SRP Section 3.7.1)

3.3 Definition of the Design-Basis Ground Motions in Japan

The criteria for the generation of the DBE ground motion in Japan consists of two parts: (1) the basic criteria utilized for the existing NPPs (pre-2006) and (2) an updated criteria that will be used in the future based on the newly revised RG (post-2006). In Japan, the terms “earthquake” and “earthquake ground motion” are carefully distinguished[6]. The S_1, S_2, and the S_S (defined below) are ground motions defined for the hazard at the site of interest. The term translated as “basic earthquake ground motion” from JEAG 4601-1987 and “Design Basis Earthquake Ground Motion” translated from NSC guidelines address the same term in Japanese “Kijun Jishindo.” Hence, the term S_1 is synonymous with either term and the terms “basic earthquake ground motion” and “Design Basis Earthquake Ground Motion” are used interchangeably herein depending of the context. Generally, throughout this report, the terms S_1, S_2, S_d, and S_S denote ground motion levels used for design and analysis.

3.3.1 Criteria in Japan Pre-2006

Pre-2006, JEAG 4601-1987 defines two deterministic earthquake ground motions used for design: the S_1 (Maximum Design Earthquake ground motion) and the S_2 (Extreme Design Earthquake ground motion). Both the S_1 and S_2 ground motions are developed deterministically based on the results of a regional investigation designed to identify important sources and their characteristics. The investigation results include the faults identified, the magnitude of each source, and the distance from the source to the site. Key elements (definitions and requirements) of the pre-2006 DBE ground motion criteria are summarized below.

3.3.1.1 S_1 – Maximum Design Earthquake Ground Motion

The scenario earthquake from which the S_1 is derived is based on “historical earthquakes” that have occurred on Class A active faults. The term “historical earthquakes” as used in Japan appears to mean earthquakes that have left evidence in the geologic record (i.e., paleoearthquakes). This differs from common usage in the United States, where historic earthquakes often denote pre-instrumental earthquakes that were documented by humans.

Class A highly active faults are defined as either faults with evidence of movement in the last 10,000 years and assessed to be capable of producing an earthquake in the near future, as defined as faults with a slip rate $\geq$ 1 mm/year or faults found to be active due to micro-earthquake activity.

The “basic earthquake ground motion,” also noted as S_1, is formulated based on a “design basis maximum earthquake” determined from the sources identified, as described above. The design-basis maximum earthquake is defined as an “earthquake selected from past earthquakes and an earthquake caused by an active fault with a frequent activity, and it has the largest effect on the site.” Based on this scenario earthquake, the S_1 ground motion is developed using deterministic techniques.

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[6] In the United States, the distinction is now also carefully made, but this was not the case in the past. As a result, terms such as “Safe Shutdown Earthquake” are still used, when what is meant is the “Safe Shutdown Earthquake ground motion,” or more accurately the “Safe Shutdown Ground Motion.” NPPs structures, like all structures, have fundamentally always been designed to ground motions and not to earthquake scenarios because it is nearly universally the rupture-induced ground motion that reaches a site that impacts the SSCs, and not the rupture itself.

3.3.1.2 S_2 – Extreme Design Earthquake Ground Motion

The S_2 is stronger than the S_1 and is formulated based on the “design basis extreme maximum earthquake” (also called the “Extreme Design earthquake”), which is defined as the deterministic motions arising from the strongest of either (i) a more onerous site-specific scenario earthquake resulting from a rarer earthquake on a less active fault, (ii) a more onerous earthquake scenario tied to a seismo-tectonic structure that has a relationship to historical earthquakes or active faults or (iii) a minimum design earthquake used throughout Japan. As translated, the extreme design scenario earthquakes may result from “earthquakes caused by an active fault, an earthquake caused by a seismic geological structure, and a shallow-focus earthquake and which has the largest effect on the site.”

The Extreme Design earthquakes can occur on active A, B and C Class faults. The scenario earthquake on a Class A fault cannot be the same scenario as forms the basis for the S_1 ground motions. Classes B and C are defined by evidence of activity in the last 50,000 years and slip rates less than 1 mm/yr. The Extreme Design earthquakes could also occur on a “seismo-tectonic structure with relationships to historical earthquakes and active faults near the periphery of the region”. Unlike the above, tectonically-based scenarios, the Extreme Design earthquake also has a minimum scenario defined as a shallow earthquake of magnitude 6.5 and hypocentral distance of 10 kilometers (km). This latter shallow earthquake acts as a minimum allowable scenario and is applied across Japan.

3.3.1.3 Ground Motion Development from the Scenario Earthquakes

Structures are always engineered against ground motions caused by earthquakes, not earthquake magnitudes. Therefore, deterministic ground motions at the site are determined based on the scenario earthquakes that were developed and characterized through the processes as described above. Maximum ground acceleration, velocity, and displacement are commonly based on empirical formulae (e.g., GMPEs) derived from recorded data and field observations. Three commonly cited authors of these attenuation relationships are Kanai, Okamoto, and Watabe.

The outputs of the GMPEs are standard pseudo-velocity response spectra [Hisada et al. 1978] that have been developed with the following attributes:

- Capable of predicting motions in the near-field, intermediate, and far-field
- Predict motions as a function of magnitude and distance
- Account for properties of the rock at the site (standard for V_s=700 m/s with an available correction factor for higher V_s values)
- Based on recorded motions, field observations, and source modeling

Figure 3-3 shows these standard response spectral shapes normalized to a PGA of 1.0 and compared to a normalized RG 1.60 horizontal response spectrum.

3.3.1.4 Other Key Elements of Definition of the S_1 and S_2

- The ground motion is defined on a control point that is at the depth of an actual or hypothetical rock outcrop. Rock is defined as material with $V_s \geq 700$ m/s.
- The relationships used to assess duration and variability of the seismic loading are empirically based.

- The ground motions are defined as horizontal components only. Vertical loading is treated statically in design.

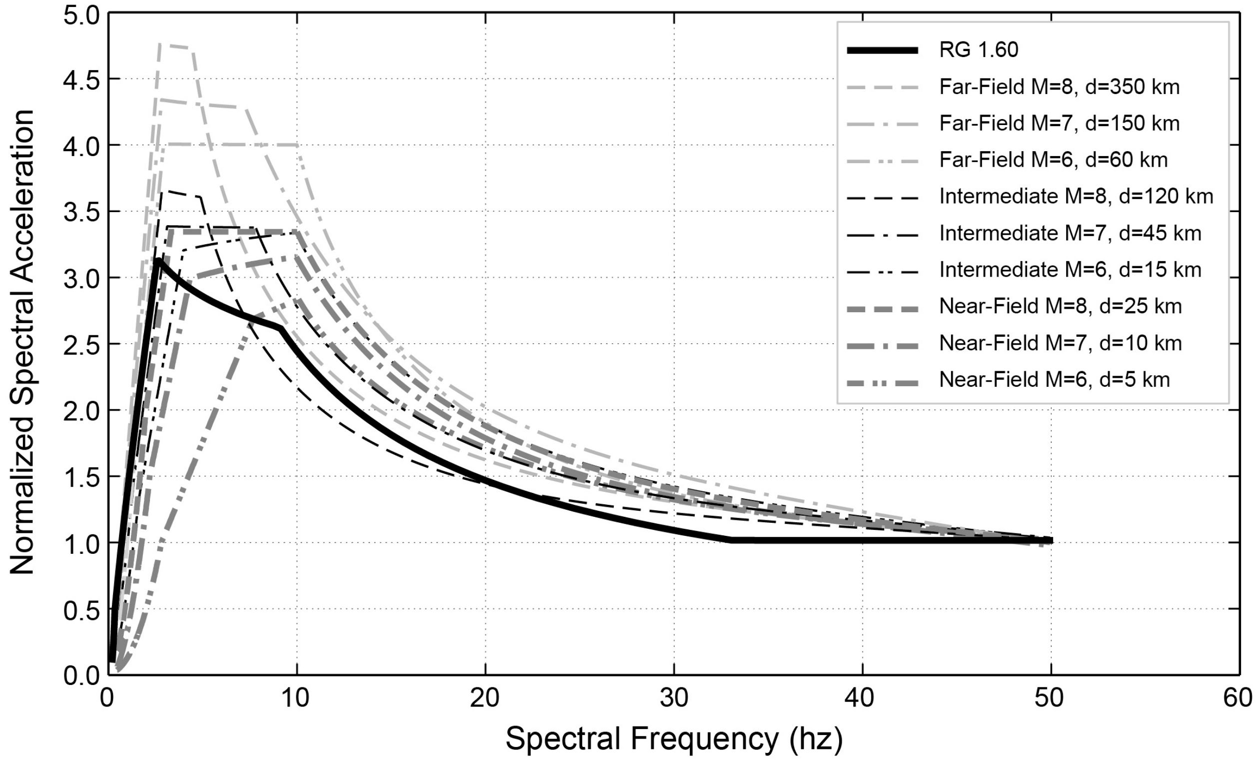

Figure 3-3 Examples of standard response spectra shapes (normalized to 1.0 at 50 Hz) for various magnitude-epicentral distance combinations at the V_s=700 m/s boundary

3.3.2 Criteria in Japan Post-2006

Importantly, the Nuclear Safety Commission (NSC) RG (as revised September 19, 2006) recognizes that there is the possibility of an earthquake producing ground motion at the site that exceeds the design-basis ground motion. The consequences of such a possibility are termed "residual risk." Appropriate attention should be paid to this possibility and to minimizing this residual risk to as low as practically possible.

NSC RG (2006) specifies two earthquake ground motions for design. These ground motion levels are designated as the Design-Basis Ground Motion, S_s, and the Elastic Design Ground Motion, S_d. As with the Pre-2006 approaches, the determination of the S_s and S_d starts with a site and regional investigation to determine sources and their relevant parameters. Key elements (definitions and requirements) of the post-2006 DBE criteria are summarized below:

3.3.2.1 S_s – Design-Basis Ground Motion

The S_s is defined by horizontal and vertical ground motions resulting from a deterministic assessment based on a scenario earthquake.

The S_s earthquake ground motion is determined by evaluating:

(i) Site-specific earthquake ground motions based on scenario earthquakes for cases where multiple seismic sources are identified. Consideration is given to earthquake source type and mechanism (intra-plate earthquakes, shallow inland earthquakes, inter-plate earthquakes), past earthquakes (size, location, etc.), fault characteristics, active faults (defined by evidence of activity in the last 120,000 to 130,000 years), and other relevant elements. The publicly available version of the new draft Nuclear Regulatory Authority (NRA) seismic design guidelines further requires consideration of activity back to 400,000 years (middle Pleistocene); if the activity is not clear, back to 120,000-130,000 years. The 120,000 to 130,000 year time frame corresponds to the late Pleistocene and is more conservative than the previous requirement of 50,000 years. The resulting set of candidate scenario earthquakes is termed "Investigation Earthquakes."

(ii) Earthquake ground motions that have been caused by sources that cannot be identified. The ground motion is developed considering past earthquake recordings for which no causative fault is identified by surface features. Response spectra are produced based on recordings and site-specific characteristics.

3.3.2.2 S_d – Elastic Design Ground Motion

The S_d is calculated by applying a ratio to the S_s. The ratio is determined based on technical evaluations of factors such as differences between safety functional limits and elastic limits, the exceedance probability of S_s, and other related information. The ratio of the S_d ground response spectrum to S_s response spectrum should not be less than 0.5.

The S_d is, therefore, derived directly from the S_s and has no direct relationship with specific earthquake sources or characteristics. It is established by engineering judgment based on the S_s.

3.3.2.3 Ground Motion Development from the Investigation Earthquakes

Design-basis ground motion at the site is determined (in part) using the sources identified as "Investigation Earthquakes" together with empirical models and fault models used as ground motion prediction tools. One set of response spectra are defined by the "Investigation Earthquakes" using appropriate GMPEs based on empirical data. A second set of response spectra are defined from dynamic fault modeling of the "Investigation Earthquakes." The fault models are important for sources near the site. The site-specific ground motion is determined based on combining the results of the empirical models and fault models. This introduces two independent methods for characterization of ground motion from the Investigation Earthquakes. The larger of the two ground motion levels is chosen. In addition, an equivalent static load, as discussed in later sections, is considered for seismic design of structures, systems, and components (SSC). The maximum of the responses either due to dynamic loading conditions or static loading conditions is selected for design/qualification of SSCs.

3.3.2.4 Other key elements of the S_s and S_d

- Appropriate methods for accounting for uncertainty should be applied taking into account the cause of uncertainty and its impact on the determination of S_s. Probabilistic concepts should be considered with an emphasis on determining the probability of exceedance of the S_s (for informational purposes).
- The ground motion is defined at a "control point." The control point is located at "the free surface of the base stratum," which is an actual or hypothetical outcrop surface. The base stratum is to be a solid foundation, defined as material with $V_s \geq 700$ m/s.
- Site-specific characteristics such as the soil profile should be taken into account.
- Duration and time variation of the ground motion should be empirically based.
- Both horizontal and vertical ground motion are to be determined.
- Design requirements for SSCs are defined based on their performance under the S_s and S_d loading levels.
  - Class S SSCs are to maintain their safety functions under the seismic forces caused by the S_s (see Section 4 for definition of Class S SSCs).
  - Class S SSCs are to be designed to the maximum of the S_d loading conditions and the static loading conditions (Sections 6 and 7). SSCs are generally to remain in the elastic range when subjected to the max (S_d, static).

Table 3-3 summarizes the comparison of the S_2 and the S_s ground motion levels for NPP sites in Japan. All of the values are specified on an outcrop of material with a value of $V_S \geq 700$ m/s. The values of S_s provided in the table are the result of new seismic hazard studies conducted after publication of the Nuclear Safety Commission (NSC) 2006 RG. Significant increases in horizontal PGA values are apparent. After the 2011 Tōhoku earthquake, NISA requires an additional reevaluation of the seismic hazard for the sites of Tomari, Onagawa, Higashidoori, Fukushima, Tokai, Hamaoka, Shika, Tsuruga, Mihama, Ooi, and Takahama. These reevaluations should reflect the lessons learned from the 2011 Tōhoku earthquake and tsunami.

Table 3-3 Original and reevaluated ground motions for NPPs in Japan (up to 2011)

Plant sites	Contributing earthquakes	New Design-Basis Ground Motion, S_s[1,2,4] (in Gal)	Original Design-Basis Ground Motion, S_2[3,4] (in Gal)
Tomari	Earthquakes undefined specifically	550	370
Onagawa	Soutei Miyagiken-oki (M8.2)	580	375
Higashidoori	Earthquakes undefined specifically	450	375
Fukushima	Earthquake near the site (M7.1)	600	370
Tokai	Earthquakes undefined specifically	600	380
Hamaoka	Assumed Tokai (M8.0), etc.	800	600
Shika	Sasanami-oki Fault (M7.6)	600	490
Tsuruga	(Mera-Kareizaki-Kaburagi F.(M7.8))	800	532
Mihama	B-Fault (M7.7)	750	405
Ooi	(Fo-A+Fo-B（M7.4))	700	405
Takahama	(Fo-A+Fo-B（M7.4))	550	370
Shimane	Shinji Fault (M7.1)	600	456
Ikata	Median Tectonic Line (M7.6)	570	473
Genkai	Takekoba Fault (M6.9)	500 (increasing to 540)	370
Sendai	Earthquakes undefined specifically	540	372
Kashiwazaki-Kariwa	F-B Fault (M7.0), Nagaoka-plain-west Fault (M8.1)	2300 (#1 side) 1209 (#5 side)	450

Notes:

[1] S_s values were finalized by NISA and NSC in October 2010. However, the NRA continues to be concerned about the possibility of displacements in the material directly under the basement of class S buildings.

[2] The ground motion levels were updated and are noted as a new S_S; however, the motions are being used principally for review. NISA was previously requiring that existing NPPs reevaluate SSCs for the updated S_S values (termed a back-check by NISA). Strengthening may have been undertaken as a result, but was voluntary. See Section 2.2.3 for a description of the back-check activities. Under the revised nuclear reactor regulatory law enacted June 2012, the backfits would be a legal requirement. NRA is now preparing a system of regulatory guidance describing the detailed requirements with a goal to publish by July 2013.

[3] S_2 was set for each unit. This table shows largest value.

[4] The control point for the ground motions is the base of the competent stratum (generally soft rock to stiff soil)

3.4 Summary of Approaches for Assessment of Design-Basis Ground Motions in the United States and Japan

3.4.1 Summary of U.S. Assessment Approaches

A summary of the three approaches that have historically been used for developing the design-basis ground motion within the United States are as shown below.

3.4.1.1 Regulatory Guide 1.60 (Pre-2007)

- Standard response spectra shape anchored to peak ground acceleration (PGA) are determined from deterministic seismic hazard study (occasionally PSHA contributed to PGA determination).
- This approach was used for existing U.S. operating NPPs (28 CEUS sites and two WUS sites).
- In some cases, spectra other than the RG 1.60 spectra were used (e.g., the Housner spectrum).

3.4.1.2 Regulatory Guide 1.165 (pre- and post-2007 up to 2010)

- SSE ground motions are determined from probabilistic seismic hazard comparison with subset of existing NPPs (Reference Probability).
- Design-basis ground motions are PSHA–based.
- The approach explicitly accounts for uncertainties.
- The approach is applicable primarily to sites in very low seismic areas.
- The RG was withdrawn April 30, 2010, and superseded by RG 1.208.

3.4.1.3 Regulatory Guide 1.208 (post-2007)

- A performance-based, risk-informed seismic hazard assessment is performed to determine the site-specific GMRS.
- The GMRS is determined using PSHA.
- The site-specific SSE is based on the GMRS that incorporates a design factor from standard ASCE/SEI 43-05.
- The approach explicitly account for uncertainties.
- The approach in RG 1.208 is used for new plant design and licensing and also for the NRC's currently ongoing hazard and risk reevaluation of operating reactors as described in [NRC 2012c].

There is also a required (10 CFR Part 50, Appendix S) minimum horizontal free-field ground motion at the building foundation level. This free-field response spectrum must be at least 0.1g PGA anchoring an appropriate response spectrum shape for the site. To date, a RG 1.60 spectrum anchored at a PGA of 0.1g has generally been used as the minimum design acceleration response spectrum.

3.4.2 Summary of Pre-2006 Assessment Approaches in Japan

- Two deterministic earthquake ground motion levels are defined and are denoted as S_1 and S_2. S_1 defines the design ground motion for Classes A_s and A SSCs. S_2 defines the functionality evaluation ground motion for Class A_s SSCs.
- S_1 and S_2 are defined on a rock outcrop (actual or hypothetical). Rock is defined as geologic materials having $V_s \geq 700$ m/s.
- Peak acceleration, velocity, and displacement are determined based on empirical formulae (no information on statistical properties of selection is available)
- Frequency content of the ground motion is defined by standard pseudo-velocity response spectra as a function of source location, magnitude, and distance. Standard spectra are defined for V_s of 700 m/s. Correction factors are available for stiffer rock with V_s up to 1,500 m/s.
- Uncertainty is not explicitly treated.
- Duration and time variation of motions are based on empirical data.
- Only horizontal components of ground motion are considered. The vertical component is treated statically.

3.4.3 Summary of Post-2006 Assessment Approaches in Japan

- The definition of an active (and in Japanese terminology also capable) fault was extended to include those with activity in the 120,000 to 130,000 years before present, instead of the earlier definition of activity in the 50,000 years before present.
- More requirements were made for site vicinity investigations to identify and characterize faulting and folding. Previously the requirement that a minimum magnitude 6.5 earthquake at 10 km be assumed meant that only the most significant tectonic structures were investigated.
- Two deterministic earthquake ground motion levels are defined: Design-Basis Earthquake Ground Motion (S_s) and Elastic-Design Ground Motion (S_d). The S_d is determined from the S_s (and should not be less than 0.5 S_s).
- "Investigation Earthquakes" are developed as scenarios from known seismic sources that could contribute to the S_s ground motion. Ground motion at the site resulting from the Investigation Earthquakes is determined using empirically-based GMPEs and from fault models (e.g., using the Green's function method). Estimates of the probability of exceedance of the site-specific ground motion resulting from the Investigation Earthquakes are requested by the regulator for informational purposes.
- In addition to the dynamic hazard analysis described above, an equivalent static load, as discussed in Section 6 for structures and Section 7 for subsystems, is considered for seismic design of SSCs. The maximum of the structure and subsystem responses due to dynamic and static loading conditions is selected for design/qualification of SSCs.
- Class S SSCs are expected to maintain their safety functions when subjected to the S_s. Class S SSCs are designed to remain elastic when subjected to the maximum of the S_d and the static applied forces.

- The existence of uncertainty is explicitly acknowledged in the new guidance and is to be taken into account by appropriate approaches. Probability of exceedance of contributing earthquakes to the S_s (and possibly S_d) is to be estimated.
- Horizontal and vertical ground motion components to be considered.
- An example of a uniform hazard response spectrum (UHRS) for a new assessment was provided to the NRC by JNES as representative of a Japanese site-specific hazard. The annual probability of exceedance of the S_s for the particular example provided was noted as between 1×10^{-4} and 1×10^{-5} per year. However, there are no criteria for probabilistic ground motions in the Japanese requirements or guidance, and so it is unclear if this ground motion probability is typical.

3.4.4 Similarities and Differences in the Design-Basis Ground Motion Criteria for Operating NPPs in Japan and the U.S.

For existing NPPs, the key points of comparison in the design-basis ground motion criteria include the following:

3.4.4.1 Peak values of ground acceleration, velocity, and displacement

- In the United States, design-basis ground motion values for operating plants were determined based on deterministic seismic hazard assessments sometimes with input from PSHA for newer plants (28 sites). The SSE ground motions for new sites are based on PSHA.
- In Japan, the S_1 and S_2 values for operating plants were based on deterministic seismic hazard analysis. The plants are assessing the S_s and S_d values, but modifications to the NPPs as a result of the assessments have been voluntary. In addition, an equivalent static load, as discussed in Section 6 for structures and Section 7 for subsystems, is considered for seismic design of SSCs. The maximum responses from the dynamic and static analyses are considered for design/qualification.

3.4.4.2 Ground response spectra shape

- The standard response spectrum (that is used to anchor a site-specific PGA) is higher in Japan than in the United States. However, a simple comparison of the standard spectra can be misleading because the true level of conservatism in the shape lies in the degree to which it captures the actual ground motion expected at a site based on the tectonic environment and style of faulting. The United States and Japan have different tectonic environments and so the relative conservatism is hard to judge. In fact, the conservatism for U.S. sites in very different tectonic regions changes. If the tectonic environments were the same, Japan would be more conservative in terms of the shape of the spectrum.
- In the United States, many operating plants are designed based on the RG 1.60 spectrum, though other spectra (e.g., the Housner spectrum) were used as well.
- In Japan, the standard response spectra shapes are from Ohsaki as documented by Hisada et al.

3.4.4.3 Impact of Changing Seismic Hazard on Nuclear Power Plant Safety

- The United States performed reevaluations in the 1990s (based on PSHA as part of the IPEEE [NRC 1991b]), which resulted in plant modifications throughout the U.S. fleet. The

United States is currently performing seismic hazard and risk reevaluation for all operating reactors and has already completed seismic walkdowns [NRC 2012c].

- Japan is currently preparing to perform PSHAs and seismic PSAs for NPPs. The PSHA and seismic probabilistic safety assessment (PSA) will address the recommendation of the NSC RG of September 19, 2006 to consider the possibility of beyond design-basis earthquake ground motions occurring denoting it “residual risk.” The approach is quantitative. For this purpose, Atomic Energy Society of Japan has developed a PSA standard [AESJ 2007], including a procedure for PSHA. PSHAs are being completed for all sites as discussed previously with the result being revised S_s ground motions and probabilistic definitions of ground motions, including ground motions that exceed the $S_{s.}$ On a parallel path, seismic PSA efforts are being initiated. However, results of the seismic PSAs will only be available in the future.

  The guidelines within JEAG 4601-1987 [NRC 1994] did not specifically address the possibility of larger than S_2 earthquake ground motions occurring (as defined in Section 3.2.1).

- To date in Japan there has been no PSHA performed and no assessment to evaluate alternative seismic hazard modeling and parameters. No risk assessment of beyond S_2 earthquake ground motions on NPPs has been performed. NSC [2006] requires reevaluation of the seismic hazard of the S_s. It is believed that the changes in the seismic hazard are evaluated for the NPPs against code criteria and not the risk-based criteria represented by seismic margin or SPRA approaches in the United States.

3.4.4.4 Treating Uncertainties in the Seismic Hazard

- U.S. practice has acknowledged and incorporated uncertainties into decisionmaking since the early 1980s. Also, the Senior Seismic Hazard Analysis Committee (SSHAC) process incorporates epistemic and aleatory uncertainties explicitly into the PSHA process.
- In Japan, uncertainty was previously incorporated into safety factors and was not acknowledged in relation to seismic hazard assessment. However moving forward, uncertainty has been acknowledged [NSC 2006] and is to be incorporated. Epistemic uncertainties are not generally dealt with explicitly but consensus of many experts and professional organizations (not necessarily nuclear) is sought for key issues.
- Generally, in the United States, there is a consistent and transparent approach to addressing uncertainties.

3.4.4.5 U.S. SSE Ground Motion and Japanese S_2 Earthquake Ground Motions

- The maximum SSE ground motion PGA of 0.67g for the San Onofre Nuclear Generating Station compares with the Kashiwazaki-Kariwa Nuclear Power Plant (KKNPP) S_2 free-field surface ground motion PGA of about 0.6g. However, the level of conservatism for any particular site depends on the actual hazard at each of these sites; and so a direct comparison cannot be made. An equivalent static load, as discussed in later sections, is considered for seismic design of SSCs in Japan. The maximum of the structure and subsystem responses due to dynamic and static loading conditions is selected for design/qualification of SSCs. The effective acceleration due to the maximum of the dynamic and static applied loading conditions is likely at or above 0.6g PGA.
- Correlation of the U.S. SSE ground motion with the Japanese S_1 and S_2 is dependent on SSC design requirements. For the U.S., SC-I SSCs are designed to the safe-shutdown earthquake (SSE) ground motion. For pre-2006 criteria, Japanese Class A SSCs would be classified as SC-I in the United States. and would be designed to the SSE ground motion

in the United States. However, in Japan, Class A SSCs are designed to the lower S_1 ground motions. Hence, one can make an assessment that the SSE ground motion is between the Japanese S_1 and S_2 when considering design requirements.

- Further, for Kashiwazaki- Kariwa Nuclear Power Plant (KKNPP), assume the S_1 PGA on the soil free-surface adheres to the same relationship as the S_1 vs. S_2 at depth (i.e., the S_1 PGA is two-thirds of the S_2 PGA). On the soil free surface, the S_1 PGA would be 0.4g. For Class A SSCs, this is closer to the operating-basis earthquake (OBE) ground motion of 0.33g than to the SSE ground motion of 0.67g. However, the impact of considering the maximum of the dynamic and static loading conditions may bring the KKNPP S_1 and the SSE ground motion closer to each other.

For fault displacement hazard and capable fault criteria, the NRC considers a fault as capable if it has shown either one movement within the past 35,000 years or multiple movements within the past 500,000 years. The Japanese criterion was similar to the first (i.e., a time frame of 50,000 years was considered for this assessment). Keeping in mind that Japan is in a seismically active region of the world and most U.S. plants are located in an intra-plate seismo-tectonic setting, the Japanese criteria could erroneously be assumed to be much more conservative. However, this is not an accurate assumption because most of the seismic sources of interest in the CEUS are characterized as source zones and not individual faults or fault zones. Source zones are characterized based on the zone activity (in terms of magnitude recurrence) and limited only by the distribution of maximum magnitudes. There is not a limitation based on the last period of activity of any particular fault. Therefore, there is no appropriate direct comparison for the CEUS. For the WUS where faults dominate the hazard, the Japanese criterion is more conservative except where multiple fault movements in the last 500,000 years are detectable in the geologic record.

3.4.5 Similarities and Difference in the Design-Basis Ground Motion Criteria for New NPPs in Japan and the U.S.

3.4.5.1 Design-Basis Ground Motion Definition

- United States
  - Performance-based derivation of risk-consistent seismic hazard (PSHA-based) is used to define the GMRS.
  - The PSHA approaches used explicitly account for uncertainties.
  - The SSE ground motion is defined in terms of site-specific GMRS and CSDRS (for certified designs). A minimum requirement for the foundation input response spectrum applies.
  - Horizontal and vertical motions are applied together.
  - Site-response analyses are required when the Vs at a site is lower than the definition of rock from the GMPE.
- Japan
  - Development of S_s includes consideration of a broad range of deterministic potential earthquake sources and uses both empirical (GMPE) and fault modeling approaches for ground motion assessment.
  - The application of the static loading conditions as an additional requirement remains in effect. This could add considerable conservatism to the procedure.

- The guidance now requires that the methods used explicitly account for uncertainties through the "residual risk" approach.
- The S_s is defined by horizontal and vertical ground motions.
- For fault displacement hazards and capable fault criteria, the NRC still uses the criteria, which considers a fault as capable if it has shown one episode of movement within the past 35,000 years and multiple movements within the past 500,000 years. The Japanese criteria now consider movement since the Late Pleistocene (120,000 to 130,000 years) as a key time frame for this assessment. The Japanese fault displacement hazard criteria have, therefore, evolved to become more conservative than originally defined over time.
- Site-specific characteristics such as the soil profile should be taken into account.
- Duration and time variation empirically based.

3.4.5.2 Design Requirements for SSCs Tied to Ground Motion Definitions

- In Japan, Class S SSCs are designed to the maximum of S_d and the static seismic force (see Sections 6 and 7). SSCs are generally to remain in the elastic range when subjected to the greater of the S_d and static loads. Class S SSCs to maintain their safety functions under the seismic forces caused by the S_s.
- In the United States, SC-I SSCs are designed to remain essentially linear under the SSE ground motions. The margin in the seismic design should be verified to assure that the HCLPF is $\geq$1.67 SSE ground motion.

4 CLASSIFICATION CATEGORIES FOR SEISMIC DESIGN

4.1 U.S. Criteria Pre-2007 and Post-2007

U.S. Nuclear Regulatory Commission (NRC) Regulatory Guide (RG) 1.29, "Seismic Design Classification," defines the structures, systems, and components (SSCs) of a nuclear power plant (NPP), including their foundations and supports, which are designated as Seismic Category I (SC-I), and must be designed to withstand the effects of the Safe Shutdown Earthquake (SSE) ground motion and remain functional. RG 1.29 was revised in 2007 and re-issued as Revision 4. Minor changes were incorporated in this latest revision. The one significant change was reference to RG 1.189, "Fire Protection for Nuclear Power Plants," Revision 1, issued March 2007, which provides guidance on seismic classification of portions of fire protection systems.

RG 1.29 includes the following SC-I I SSCs:

- Reactor coolant pressure boundary
- Reactor core and reactor vessel internals
- Emergency core cooling, post-accident containment heat removal, post-accident containment atmosphere cleanup
- Systems required for reactor shutdown, residual heat removal, and spent fuel pool cooling
- Portions of the steam systems of boiling water reactors (BWRs) extending from the outermost containment isolation valve up to but not including the turbine stop valve, and connected piping of a nominal size of 6.35 cm (2 ½ in.) or larger, up to and including the first valve that is either normally closed or capable of automatic closure during all modes of normal reactor operation (the turbine stop valve should be designed to withstand the SSE and maintain its integrity)
- Steam and feed water system of pressurized-water reactors (PWRs) from the secondary side of the steam generators to and including the outermost containment isolation valves, and connecting piping of 6.35 cm (2 ½ in.) or larger, up to and including the first valve that is either normally closed or capable of automatic closure during all modes of normal reactor operation
- Cooling water, component cooling, and auxiliary feedwater systems, including the intake structure, that are required for emergency core cooling, post-accident containment heat removal, post-accident containment cleanup, residual heat removal, and spent fuel cooling
- Systems that are required to supply fuel for emergency equipment
- Electrical and mechanical devices and circuitry between the process and the input terminals of the actuator systems involved in generating signals that initiate protective action
- Systems required for monitoring of systems important to safety and for actuation of systems important to safety
- Spent fuel pool structure, including fuel racks
- Reactivity control system (e.g., control rods, control rod drive systems, and boron injection systems)

- Control room and its associated equipment, and all equipment needed to maintain the control room within safe habitability limits for personnel and safe environmental limits for vital equipment
- Primary and secondary reactor containment
- Systems, other than radioactive waste management systems not covered above, that contain radioactive material whose postulated failure would result in offsite doses that are more than 5 millisieverts (0.5 rem) to the whole body
- Class 1E electrical systems that provide emergency electrical power needed for functioning of plant features described above

RG 1.29 specifies that any SSCs whose function is not required but whose failure could reduce the functioning of any of the systems described above to an unacceptable safety level, or could result in incapacitating injury to occupants of the control room, should be designed so that the SSE would not cause such failure (spatial systems interactions). Often, these SSCs are designated as Seismic Category II (SC-II) by licensees.

Pre-2007, SC-I SSCs were designed to the operating basis earthquake (OBE) and the SSE ground motions. Post-2007, requirements to design to the OBE have been removed; in fact, if the OBE ground motion is less than or equal to one-third of the SSE ground motion, no additional seismic analysis is required and it is assumed that the SSE design is sufficient. NRC Standard Review Plan (SRP) [NUREG-0800] Section 3.2.1, “Seismic Classification,” itemizes NRC’s acceptance criteria for submittals by applicants. SRP Section 3.2.1 refers to RGs 1.29, 1.151, 1.143, and 1.189 for more detailed information. If the OBE is set to one-third of the SSE, SRP Section 3.2.1 requires submittal of a list of SSCs necessary for safe operation after an OBE ground motion occurs at the site.

4.2 Japan Criteria Pre-2006 and Post-2006

Pre-2006, Japan Electric Association (JEA) Standard JEAG (Japan Electric Association Guidelines) 4601-1987 itemized four categories for seismic classification of SSCs according to their function and identified typical equipment that fall into each category. The four categories include:

Class A_s	SSCs that may cause loss of coolant if damaged. SSCs that are required for emergency shutdown of the nuclear reactor and are needed to maintain the shutdown state of the reactor in a safe state; facility for storage of spent fuel; and nuclear reactor containment
Class A	SSCs that are needed to protect the public from the radioactive hazard in the case of a nuclear reactor accident, and SSCs, the malfunction of which may cause radioactive hazard to the public, but that are not classified as Class A_s.
Class B	SSCs that are related to the highly radioactive substance, but are not classified as Class A_s or A
Class C	SSCs that are related to the radioactive substance, but are not classified in the above seismic classes, and facilities not related to radioactive safety.

The above functional categories are further broken down in JEAG 4601-1987 Table 5.1.2.1 (reproduced here as Table 4-2). An important aspect of the seismic classifications A_s and A is the associated seismic design requirements. Class A_s and A SSCs are designed to the maximum of the S_1 earthquake and an equivalent static load. However, Class A_s SSCs are further required to maintain their safety function when subjected to the S_2 earthquake ground motion. Table 4-1 itemizes the U.S. and Japanese seismic classification and resulting seismic design criteria for selected systems with emphasis on the Class A_s and A SSCs.

Table 4-1 Seismic design requirements for selected system

Typical System	U.S. Seismic Category I	JEAG Class A_s	JEAG Class A	JEAG Class B
Primary Coolant System	OBE + SSE	S_1, S_2		
Control Rod Drive	OBE + SSE	S_1, S_2		
Reactor Internals	OBE + SSE	S_1, S_2		
Residual Heat Removal	OBE + SSE	S_1, S_2		
Emergency Core Cooling	OBE + SSE		S_1	
Containment Spray and Cooling	OBE + SSE		S_1	
Heating, Ventilating, and Air Conditioning for Engineered Safety Feature	OBE + SSE		S_1	
Spent Fuel Pool and Racks	OBE + SSE	S_1, S_2		
Normal Spent Fuel Pool Cooling	OBE + SSE			Static Seismic Force
Emergency Spent Fuel Pool Cooling			S_1	

A comparison of the RG 1.29 list of SC-I SSCs with the list in Table 4-2 (a translation of JEAG Table 5.1.2.1) generally shows that the combination of Japan Classes A_s and A correspond to U.S. SC-I SSCs. The exception being that systems that mitigate a loss-of-coolant accident (LOCA) are not designed for the S_2 earthquake ground motion as specified in JEAG 4601-1987.

In terms of seismic classification, the revised Nuclear Safety Commission (NSC) RG (September 19, 2006) specifies a new classification denoted as Class S, which combines Class A_s and A SSCs (pre-2006) and some Class B equipment. Thus, the inconsistency between the U.S. and Japan classification noted above is eliminated in the Post-2006 Japanese seismic code revision. In addition, Class S SSCs will be designed to two earthquake ground motion levels designated, S_s and S_d that also, required merging of Classes A_s and A. Design-basis ground motions S_s and S_d are discussed in Section 3.2. Classes B and C retain the same definition as in Pre-2006 guidance.

Table 4-2 Reproduction of translation of JEAG 4601-1987 Table 5.1.2-1(a) for seismic categorization of structures, systems, and components

<table>
<tr><th colspan="5">JEAG 4601-1987 Definitions of aseismic importance and facilities of various classes(1)</th></tr>
<tr><th rowspan="2">Classification and definition of seismic importance</th><th rowspan="2">Classification of functions</th><th>Primary equipment</th><th colspan="2">Indirect support structures</th></tr>
<tr><th>Application range</th><th>Application range</th><th>Seismic ground motion</th></tr>
<tr><td rowspan="4">Class A_s:
Parts, damage of which may cause loss of coolant; parts that are required for emergency shutdown of the nuclear reactor and are needed to maintain the shut-down state of the reactor in a safe state; facility for storage of spent fuel; and nuclear reactor containment</td><td>(i) Piping and equipment that form the “pressure boundary of nuclear reactor coolant” (as defined in “Guidelines of safety design in evaluation of light water reactor facilities for power generation”)</td><td>(1) Pressure containment of nuclear reactor (B)
(2) Containment of nuclear reactor (P)
(3) Containments, piping, pumps, and valves belonging to the pressure boundary of nuclear reactor coolant</td><td>(1) Reactor building
(2) Control building
(3) Pedestal of pressure containment of nuclear reactor (B)
(4) Internal concrete (P)
(5) Auxiliary building (P)</td><td>S_2</td></tr>
<tr><td>(ii) Equipment for storage of spent fuel</td><td>(1) Spent fuel pool (B)
(2) Spent fuel storage rack (B)
(3) Spent fuel pit (P)
(4) Spent fuel rack (P)</td><td>(1) Reactor building
(2) Auxiliary building (P)
(3) Fuel handling building (P)</td><td>S_2</td></tr>
<tr><td>(iii) Equipment used for applying rapid negative reactivity for emergency shutdown of nuclear reactor, and equipment for maintaining the shutdown state of the nuclear reactor</td><td>(1) Control rods, control rod driving unit, and control rod driving hydraulic system (the portion related to the scram function) (B)
(2) Control rod cluster and control rod driving unit (the portion related to the scram function) (P)
(3) Boric acid injecting unit (transfer system) (P)</td><td>(1) Reactor building
(2) Internal concrete (P)
(3) Auxiliary building (P)
(4) Control building (P)
(5) Diesel building (P)</td><td>S_2</td></tr>
<tr><td>(iv) Equipment for removal of decay heat from the reactor core after shutdown of the nuclear reactor</td><td>(1) Cooling system for isolating reactor (B)
(2) High-pressure reactor core spray system (B)
(3) Residual heat removal system (equipment required for cooling mode operation in shutdown state) (B)
(4) Suppression pool as cooling water source (B)
(5) Main steam feedwater system (primary feedwater check valve, to secondary side of steam generator, to</td><td>(1) Reactor building
(2) Control building
(3) Foundation of seawater pump, and other structures for supporting the seawater system (for emergency cases) (B)
(4) Internal concrete (P)
(5) Auxiliary building (P)
(6) Diesel building (P)
(7) Foundation of seawater pump and other structures for supporting the seawater system (P)</td><td>S_2</td></tr>
</table>

JEAG 4601-1987 Definitions of aseismic importance and facilities of various classes[1]				
Classification and definition of seismic importance	Classification of functions	Primary equipment	Indirect support structures	
		Application range	Application range	Seismic ground motion
		main steam isolating valve) (P) (6) Auxiliary feedwater system (P) (7) Condensate water tank (P) (8) Residual heat removal (P)		

Table 4-2 Reproduction of translation of JEAG 4601-1987 Table 5.1.2-1(a) for seismic categorization of structures, systems, and components (Continued)

JEAG 4601-1987 Definitions of aseismic importance and facilities of various classes[1]				
Classification and definition of seismic importance	Classification of functions	Primary equipment	Indirect support structures	
		Application range	Application range	Seismic ground motion
Class A_S (cont.)	(v) Equipment that becomes a pressure barrier for preventing direct discharge of radioactive substances in case of accidental rupture of the coolant pressure boundary of the nuclear reactor	(1) Containment structure[2] (2) Piping and valves belonging to the containment boundary of the nuclear reactor	(1) Reactor building (2) Auxiliary building (P)	S_2
			(1) Control building (2) Diesel building (P) (3) Reactor building	S_1
Class A: Parts, which are needed to protect the public from the radioactive hazard in the case of a nuclear reactor accident, and parts, malfunction of which may cause radioactive hazard to the public, but are not classified as Class A_s	(i) Equipment required for removing decay heat from reactor core after accidental rupture of the coolant pressure boundary of the nuclear reactor	(1) Emergency core cooling system (B) • High-pressure core spray system • Low-pressure core spray system • Residual heat removal system (equipment required for operation in the low-pressure core water injection mode) • Automatic pressure relief system (2) Suppression pool as cooling water source (B) (3) Safety injection system (P) (4) Emergency core cooling system (ECCS) (P) (5) Water tank for exchange of fuel (P)	(1) Reactor building (2) Control building (3) Foundation of seawater pump and other structures for supporting the seawater system (for emergency use) (B) (4) Auxiliary building (P) (5) Diesel building (P) (6) Foundation of seawater pump and other structures for supporting the seawater system (P)	S_1

	(ii) Equipment, not included in aseismic Class As (V), for preventing release of radioactive substances to the outside in an accident accompanied by leakage of radioactive substances	(1) Residual heat removal system (equipment required for cooling containment and for operation in spray mode) (B) (2) Reactor building (B) (3) Combustible gas concentration control system (B) (4) Emergency gas treatment system and exhaust port (B) (5) Nuclear reactor containment pressure-suppressing equipment (diaphragm floor, vent pipe) (B) (6) Main steam separating valve leakage control system (B)	(1) Reactor building (2) Control building (3) Foundation of seawater pump and other structures for supporting seawater system (for emergency use) (B) (4) Primary exhaust pipe (B) (In case of support of exhaust port of emergency gas treatment system)	S_1

Table 4-2 Reproduction of translation of JEAG 4601-1987 Table 5.1.2-1(a) for seismic categorization of structures, systems, and components (Continued)

JEAG 4601-1987 Definitions of aseismic importance and facilities of various classes(1)				
Classification and definition of seismic importance	Classification of functions	Primary equipment	Indirect support structures	
		Application range	Application range	Seismic ground motion
Class A (cont.)	(ii) (cont.)	(7) Suppression pool as cooling water source (B) (8) Containment spray system (P) (9) Water tank for replacement of fuel (P) (10) Annulus seal (P) (11) Annulus air cleaner (P) (12) Containment gas exhaust pipe (P) (13) HVAC for auxiliary safety equipment room (P) (including engineering safety facilities)	(5) Auxiliary building (P) (6) Reactor containment vessel (P) (7) External shield (P) (8) Diesel building (P) (9) Foundation of seawater pump and other structures supporting the seawater system (P)	S_1
	(iii) Others	(1) Fuel pool water feed equipment (for emergency use) (B) (2) Boric acid solution injecting system (B) (3) Spent fuel pool feed equipment (for emergency use) (P)	(1) Reactor building (2) Auxiliary building (P) (3) Fuel handling building (P)	S_1
		Internal structures of reactor	(1) Reactor pressure containment pedestal (B) (2) Reactor building (B)	S_2

Class B: Parts, which are related to the highly radioactive substance, but are not classified as Class A_s and Class A	(i) Equipment that contains or can contain primary coolant in direct contact with coolant pressure boundary of nuclear reactor	Main steam system (from outside main steam isolation valve to turbine primary blockage valve) (B) [4]	(1) Reactor building (2) Turbine building (portion for supporting the piping and valves from outside main steam isolation valve to primary blockage valve) (B)	S_1
		(1) Main steam system and feedwater system (B) (2) Reactor coolant purification system (B) (3) Extraction system and residue extraction in chemical volume control system (P)	(1) Reactor building (2) Turbine building (B) (3) Auxiliary building (P) (4) Internal concrete (P)	S_B[5]

Table 4-2 Reproduction of translation of JEAG 4601-1987 Table 5.1.2-1(a) for seismic categorization of structures, systems, and components (Continued)

JEAG 4601-1987 Definitions of aseismic importance and facilities of various classes[1]				
Classification and definition of seismic importance	**Classification of functions**	**Primary equipment**	**Indirect support structures**	
		Application range	**Application range**	**Seismic ground motion**
Class B (cont.)	(ii) Equipment for containing radioactive waste, excluding those that have a small content or a special storage method, therefore possess a smaller radioactive effect to the public in case of rupture than the annual exposure dose allowable outside the peripheral monitoring region	Equipment for processing wastes, excluding that belonging to Class C	(1) Waste treatment building (2) Reactor building (P) Auxiliary building (P)	S_B
	(iii) Equipment which is related to radioactive substances other than the radioactive waste, and the rupture of which may cause an excessive radioactive exposure to the public and employees	(1) Shields with significant effect in reducing the radiation level (2) Steam turbine, condenser, feedwater heater, and major piping (B) (3) Condensing/desalting equipment (B) (4) Condensate storage tank (B) (5) Fuel pool purifying system (B) (6) Control rod drive hydraulic system (the portion containing radioactive fluid) (B) (7) Reactor building crane (B) (8) Fuel handling equipment (B) (9) Control rod storage rack (B) (10) Spent fuel pool purifying system (P) (11) Parts other than Class C in the chemical volume control system (P) (12) Auxiliary building crane (P) (13) Spent fuel pool crane (P) (14) Fuel exchange crane (P) (15) Fuel transfer equipment (P)	(1) Reactor building (2) Turbine building (B) (3) Turbine pedestal (B) (4) Internal concrete (P) (5) Auxiliary building (P)	S_B

Table 4-2 Reproduction of translation of JEAG 4601-1987 Table 5.1.2-1(a) for seismic categorization of structures, systems, and components (Continued)

<table>
<tr><th colspan="5">JEAG 4601-1987 Definitions of aseismic importance and facilities of various classes(1)</th></tr>
<tr><th rowspan="2">Classification and definition of seismic importance</th><th rowspan="2">Classification of functions</th><th>Primary equipment</th><th colspan="2">Indirect support structures</th></tr>
<tr><th>Application range</th><th>Application range</th><th>Seismic ground motion</th></tr>
<tr><td rowspan="2">Class B (cont.)</td><td>(iv) Equipment for cooling spent fuel</td><td>(1) Fuel pool cooling system (B)
(2) Spent fuel pool cooling system (P)</td><td>(1) Reactor building
(2) Auxiliary building (P)
(3) Fuel handling building (P)
(4) Foundation of seawater pump and other structures supporting the seawater system (P)</td><td>S_B</td></tr>
<tr><td>(v) Equipment not belonging to aseismic Class A_s and Class A, and is used to suppress dissipation of radioactive substances to the outside when the radioactive substances are released</td><td></td><td></td><td>S_B</td></tr>
</table>

Notes:

(1) Courtesy "JEAG 4601, Supplment-1984", with the contents reorganized in this table.

(2) In principle, there is no need to perform evaluation using basic earthquake ground motion S_2. However, as it is the final barrier for preventing dissipation of the radioactive substance, only the reactor containment boundary is taken as aseismic Class As. For the isolating value, the requirement is that it should maintain as isolated state after basic earthquake ground motion S_2 takes place.

(3) The CAD scheme is also included.

(4) Although it belongs to aseismic Class B, analysis should be performed to ensure no failure after basic earthquake ground motion S_1.

(5) S_B is the seismic input to be applied for aseismic Class B equipment.

HVAC is heating, ventilation, and air conditioning

(B): BWR

(P) PWR

no mark: common to BWR, PWR

4.3 Summary of U.S. and Japan Seismic Classification Systems and Related Seismic Design Criteria

Pre-2007 and Post-2007 elements of the U.S. seismic classification and design criteria include three seismic categories: SC-I, SC-II, and non-Seismic. The classes are designed to the following criteria:

- SC-I SSCs are designed to OBE and SSE ground motions (Pre-2007) and the SSE ground motion (Post-2007)
- SC-II SSCs are designed to prevent damage or failure causing loss of function of SC-I SSCs in under seismic loading
- For non-Seismic SSCs, the seismic design is to industrial standards

Post-2007, elements of the U.S. seismic classification and design criteria also include:

- Seismic considerations for fire suppression systems
- Listing of SSCs needed for safe operation after OBE

Pre-2006 (JEAG 4601-1987) key elements of Japanese seismic classification and design criteria include:

- Class A_s SSCs are designed to the maximum equivalent static loading condition and the dynamic loading condition (S_1). They must maintain the ability to perform their safety function under the S_2 ground motion level
- Class A SSCs are designed to the maximum of the equivalent static loading condition and the dynamic loading condition (S_1).
- Classes B and C SSCs are designed to the equivalent static loads.

Post-2006 (NSC RG 2006) key elements of Japanese seismic classification and design criteria include:

- Class S SSCs are designed to the maximum of the equivalent static loading condition and the dynamic loading condition from the S_d ground motion. They are designed to essentially remain in the elastic range of response.
- Class S SSCs are designed to maintain their safety functions under the seismic forces caused by the S_s ground motion.
- Classes B and C SSCs are designed to the equivalent static load

The key comparisons between the seismic classification and design criteria in Japan and the United States are summarized below and in Table 4-1.

- Japan Classes A_s and A (Pre-2007) or Class S (Post-2006) (possibly with some items from Class B) are comparable to U.S. SC-I with the exception of the loss-of-coolant accident (LOCA) mitigating equipment for Japan's Pre-2006 criteria. Post-2006 this exception has been eliminated.
- U.S. design loading conditions for SC-I SSCs are the OBE and the SSE ground motions (Pre-2007), and only the SSE ground motion (Post-2007). The Japanese standard requires only the S_1 (equivalent to the OBE ground motion) earthquake design for Class A (Pre-2006) SSCs.

- U.S. designs are required to be evaluated to beyond design-basis earthquake ground motions. Current standard plants require applicants to demonstrate High Confidence of Low Probability of Failure (HCLPF) margin of 1.67 times design (SECY-93-087).[1]
- The Japanese Post-2006 NSC RG recognizes the existence of “residual risk” and specifies it should be minimized. This minimization of risk should involve an evaluation (qualitative or quantitative and could potentially include a beyond design-basis evaluation) to determine its acceptability.

[1] Additionally, the NRC’s IPEEE program investigated implications of beyond-design-basis events.

5 SOIL-STRUCTURE INTERACTION AND STRUCTURAL RESPONSE

5.1 U.S. Criteria Pre-2007 and Post-2007

The U.S. Nuclear Regulatory Commission (NRC) Standard Review Plan (SRP) Sections 3.7.1 and 3.7.2 state acceptable approaches to calculating seismic response of structures including soil-structure interaction (SSI).

SRP Section 3.7.1 Seismic Design Parameters, Revision 3, March 2007

SRP Section 3.7.1 deals principally with the determination of the ground motion for the various cases of interest. Previously, the definition of the ground motion in terms of ground response spectra was discussed. SRP Section 3.7.1 provides guidelines on the use of time histories in the seismic analysis of the main structures, including SSI or for fixed-base analysis.

SRP Section 3.7.2 Seismic System Analysis, Revision 3, March 2007

SRP Section 3.7.2 provides acceptable approaches to the seismic analysis of systems including the effects of SSI. Many topics are presented and discussed. Those topics most relevant to this effort are:

- Seismic Analysis Methods – Dynamic Analysis Method
    - Appropriate dynamic analysis methods should be used for the problem being addressed and the end quantities of interest (e.g., forces/moments, in-structure response spectra).
    - Three components of ground motion are used as inputs (two horizontal and the vertical).
    - Translation, torsion, and rocking responses of the structure and foundation should be considered.
    - Discretization of the structure model must be adequate to model the frequency range of interest and the end quantities of interest.
    - Consideration is given to relative motions between adjacent supports of Seismic Category I (SC-I) structures, systems, and components (SSCs).
    - Other significant effects (e.g., hydrodynamic loads) should be included in the analysis.
- Seismic Analysis Methods - Equivalent Static Load Method
    - The seismic analysis method is considered to be a conservative approach.
    - The method is typically applied to simple structures.
- Systems versus subsystems
    - SC-I structures assessed in conjunction with the foundation and its supporting media (soil or rock) are defined as “seismic systems.” Other SC-I SSCs that are not designated as “seismic systems” should be considered as “seismic subsystems.”
- Decoupling criteria for systems and subsystems should be applied.

- Modeling of structures can be achieved through different approaches, including lumped mass stick models, finite element method (commonly called FEM) models, and two-stage analysis (dynamic model for overall response, detailed static model for structure design).
- Soil-structure interaction
    - Fixed-base analysis is used for structures founded on materials with a $V_s \geq 2438$ m/s (8000 ft/s).
        - Input motion comes from the standard response spectra (i.e., the Certified Seismic Design Response Spectra and/or site specific Ground Motion Response Spectra). Three components of motion (two horizontal and the vertical) are used.
    - Soil profiles are developed to address behavior, variability, and uncertainty.
        - Equivalent linear shear moduli and soil material damping (≤15 percent) is determined as a function of ground motion.
        - Three profiles (Best estimate (BE), Lower bound (LB), and Upper bound (UB)) are used in the SSI analyses.
        - Minimum soil property variation (COV = 0.5) is used with the BE profile to develop the UB and LB profiles using the following equations:

          LB = BE / (1 + COV)

          UB = BE x (1 + COV)
    - The foundation-level free-field horizontal response spectrum is required to be at least 60 percent of the surface motion. The envelope of the three soil cases can satisfy this requirement.
    - Sensitivity studies are performed to identify the potential for other issues, such as separation of structure/foundation from the soil, including uplift and sliding.
- Seismic responses (e.g., forces, moments, accelerations, and in-structure response spectra) are developed for use in SSC design and qualification
    - The envelope of the responses from the three soil cases is used for SSC design and qualification.
    - The in-structure response spectra (ISRS) is developed using the following:
        - The envelope of three soil cases is used.
        - The peaks of ISRS are broadened ±15 percent.
        - The ISRS is smoothed and valleys filled in.

  Regulatory Guide (RG) 1.61 defines allowable damping values for structures and subsystems as a function of construction type and expected stress level during earthquake excitations. The values listed for the SSE are applicable and can be used for structure design for the SSE without further justification. For defining input motion to subsystems, SSE damping values may only be used if justified based on stress levels expected during the SSE ground motion. For lower expected stress levels, reduced values of damping should be used to generate input to subsystems.

- SSI analysis methods are currently used by industry

The revision to standard American Society of Civil Engineers (ASCE), "Seismic Analysis of Safety-Related Nuclear Structures and Commentary," Standard ASCE 4-98, 1998 [ASCE 4-98] (currently under development with anticipated publication in 2013) extensively expands the discussion of SSI analysis methods with a focus on substructure methods, especially methods such as A Linear Continuum Mechanics Approach to Soil-Structure Interaction (CLASSI) and System for Analysis of Soil-Structure Interaction (SASSI), which are by definition linear (or equivalent linear). In addition, the current revision of ASCE 4 introduces nonlinear SSI analysis methods in Appendix B with the clear expectation that nonlinear SSI analysis techniques will evolve over the next few years and become one of the preferred analysis methods for the phenomena.

Substructuring methods can be categorized according to their approach to solving the SSI problem. Three methods of varying analysis capability and limitations are described below.

SASSI –System for Analysis of Soil-Structure Interaction

SASSI evaluates the dynamic response of two- and three-dimensional foundation-structure systems. SASSI uses linear finite element modeling and the frequency domain solution methods. The soil is modeled as a uniform or horizontally layered, elastic or viscoelastic medium overlying a uniform half-space. The soil material model is based on complex moduli, which produces frequency-independent hysteresis damping. The structures are modeled by two- or three-dimensional finite elements interconnected at node points. Seismic input motion is defined by acceleration time histories and may be assumed to be comprised of vertically incident or inclined body waves or surface waves. Two basic methods of analysis in SASSI are called the "flexible volume" and "subtraction"[1] methods. These methods are formulated in the frequency domain. SASSI may be used for foundation-structure systems and is capable of modeling flexibility of the foundation. Generally, horizontal and vertical models are analyzed independently and the results combined post analyses.

Advantages include the ability to model complex foundation geometry, foundation embedment, and foundation flexibility. Limitations are primarily resource based (i.e., the lack of the ability to analyze very detailed structure models in a timely manner and the difficulty in easily performing sensitivity studies).

CLASSI – Soil-Structure Interaction: A Linear Continuum Mechanics Approach

CLASSI solves the SSI problem by making use of the below attributes.

- Free-field ground motion is defined by acceleration time histories defined at the control point. Vertically incident or non-vertically incident shear and dilatational body waves may be specified as the wave propagation mechanism. Also, surface waves may be specified.
- The soil profile is modeled as semi-infinite layers overlying a half space. The material properties of the layers and half space may be modeled as elastic or viscoelastic assuming a complex moduli representation.

[1] The subtraction method continues to be evaluated due to anomalies identified in its application. Caution is recommended in its application. [DOE 2011, Anderson 2011].

- The geometry of the foundation is defined and discretized.
- Complex-valued, frequency-dependent Green's functions for horizontal and vertical point loads are generated; hence, the term "continuum mechanics" in the method's name.
- Foundation input motion is defined as the response of a rigid, massless foundation subjected to the free-field ground motion. This portion of the problem is termed "kinematic interaction."
- The foundation impedance matrix is calculated as a complex-valued, frequency-dependent force displacement matrix relating forces and moments on the foundation to displacements and rotations of the foundation.
- Detailed fixed-base finite element structure models are developed, eigenvalue extractions performed, and dynamic characteristics are projected to the foundation for SSI analysis.
- CLASSI combines all of the above elements to solve for response on the foundation resulting from SSI. In general, the result is six components of response on the foundation. CLASSI then solves for responses in the structure subjected to the six components of foundation motion. Each direction of free-field motion may be analyzed independently or all three may be analyzed simultaneously.

Advantages include the ability to analyze very detailed structure models using the results from an eigenvalue extraction performed using the structure analysis program of choice. SSI analyses performed by CLASSI are computationally efficient allowing sensitivity studies to be easily performed. Intermediate steps produce results that are easy to validate. It is an excellent method to use when performing probabilistic SSI response analyses. Limitations of the standard version of CLASSI are the ability to only model foundations that behave rigidly and are surface founded.

Hybrid method

A hybrid method using the Green's functions from SASSI, generated for embedded foundations, and applied to the methodology of CLASSI to generate the foundation input motion and the foundation impedances taking into account embedment provides a very effective alternative [Johnson 2010] approach.

Advantages of the hybrid method include benefits from SASSI and CLASSI. The benefits are the same as those of SASSI for modeling embedded foundations and partial structures. The benefits of CLASSI in the hybrid method include the ability to model very detailed structure models in a computationally efficient SSI analysis (including the performance of sensitivity studies). Validation of intermediate steps is easily accomplished.

Additional Simplified Method in ASCE 4

It is important to note that one key limitation of all of the methods described above is that they only solve the linear SSI problem.

For this reason, an additional method called the simplified soil spring method is retained in ASCE 4. Historically, this method has been applied only to structures founded on basemats of regular geometry and assumed to behave rigidly. Foundation impedances are simplified and generally are assumed to be frequency-independent. For these conditions, conventional dynamic analysis programs may be used to generate structure response.

Advantages to the additional substructure method in ASCE 4 are: (i) nonlinear SSI analyses can be performed for these simpler representations of the foundation-soil behavior (and so nonlinear behavior of the soil springs or of the structure can be treated) and (ii) it is a cost effective approach to model the effects of SSI for simple structures. Limitations are that it models the foundation-soil system very simply, which may not be a valid approximation for actual soil-structure configurations.

In addition, direct numerical simulation methods may be employed for linear and nonlinear systems. Linear methods most often solve the problem in the frequency domain. Nonlinear methods use the time domain[2].

5.2 Japanese Criteria Pre-2006 and Post-2006

The Post-2006 Japanese criteria (Nuclear Safety Commission (NSC) RG) are not detailed enough to define specific differences between Pre-2006 (in the Japan Electric Association (JEA) guidance document JEAG 4601-1987) and Post-2006 criteria except in a few instances as discussed later in this section. Guidelines for the NSC RG 2006 have been implemented in revisions to JEAG-4601, which are denoted JEAC 4601 (C denotes Code) and JEAG 4601 (G denotes Guidelines). These documents have been completed and issued in Japanese. JEAG 4601-1987 is a very detailed and comprehensive "handbook" in the areas of structure modeling and SSI; and the guidance discusses the elements mentioned here.

For Pre-2006, SSI and structure response analyses are separated into two separate cases. One of which uses the S_1 design-basis ground motion and one of which uses the S_2 design-basis ground motion. In general terms, SSI and structure response for the S_1 ground motion is treated linearly for the structure modeling and equivalent linearly for modeling of the soil or rock elements. SSI and structure response for the S_2 earthquake ground motion is treated nonlinearly. Two aspects of nonlinear behavior are required to be modeled. These aspects are nonlinear behavior of the structure and uplift and sliding of the foundation during SSI. Consideration of the S_2 is required only for those SSCs designated Class A_s.

S1 Linear Analysis

The key elements of the S_1 ground motion-based SSI and structure response analysis in the Japanese guidance are:

- Definition of the input motion
  - Input motion is provided as response spectra and/or earthquake recordings
  - Input motion is defined on a rock outcrop where rock is defined as material with Vs ≥700 m/s
  - Free-field motion is propagated to other locations in the soil column by one-dimensional wave propagation theory

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[2] The NRC is currently developing a computational tool for non-linear time domain analysis [Jeremic 2011]

- Horizontal components are treated dynamically (the vertical component is treated as constant static force)

- Definition of the soil/rock profile
  - Field and in-situ tests are used to determine soil profile, low strain properties, and strain dependent properties
  - Equivalent linear soil properties are used in the SSI analysis (BE is assumed)
- SSI models and analysis (description of methods and overall requirements)
  - Soil spring methods (frequency-dependent and –independent) are defined
  - Finite element methods or lumped mass lattice models are defined
  - Models must match soil/rock/structure configuration
  - Significant effort is expended to verify SSI models through scale model field tests and full scale structure response measurements
  - BE soil properties are used and there is no variation in soil properties considered to account for uncertainty in soil properties and SSI modeling
- Structure models
  - To the extent possible, structures are designed to be "rigid"
  - Detail of models should match analysis end objectives
  - Stiffness properties come from:
    - Architectural Institute of Japan guidelines
    - Test results for structure components
  - Damping properties are defined in JEAG 4601 (see Table 5-1 below)
- Seismic responses (e.g., forces, moments, accelerations, and ISRS) for SSC design and qualification are developed from structure response
  - Calculated for one soil case only
  - In-structure response spectra (ISRS)
    - The ISRS peak is broadened ±10 percent
    - The ISRS is smoothed to remove peaks and valleys

Table 5-1 Damping values from NRC RG 1.61 and JEAG 4601-1987

Structure Material	RG 1.61	JEAG 4601-1987
	SSE	S_1 (elastic range)
Reinforced Concrete	7%	5%
Reinforced Masonry	7%	NA
Prestressed Concrete	5%	3%
Welded Steel or Bolted Steel with Friction Connections	4%	See below
Bolted Steel with Bearing Connections	7%	See below
Steel Containment	4%	1%
Steel Frame Building Structure	4 to 7%	2%
Bolted/Riveted Structure	4 to 7%	2%
Note: Values shown are from NRC RG 1.61 Revision 1, 2007		

***S_2* Nonlinear Analysis**

The general aspects of the definition of the input motions and the soil profile are the same as described in S_1 with appropriate modifications in the details reflecting the S_2 earthquake ground motion.

Other key elements of the S_2 SSI and structure response analysis are:

- SSI models and analysis
  - Model is capable of including and assessing uplift and sliding
  - Radiation and material damping are included
  - Models must match soil/rock/structure configuration
  - Significant effort is expended to verify SSI models through scale model field tests and full scale structure response measurements
  - BE soil properties are used with no variation in soil properties to account for uncertainty in soil properties and SSI modeling
  - Preferred method appears to be nonlinear soil springs and lumped mass/stick models with nonlinear structure elements
- Structure models
  - Stiffness modeling is based on experimental data of structural elements
  - Energy dissipation occurs through hysteresis loops
- Seismic responses
  - Appropriate quantities are determined to demonstrate that safety margin exists based on structure ultimate strength
  - Results are used to ensure the response of the supporting building/structure does not degrade the functioning of the systems

Japan Criteria Post-2006

NSC RG 2006 specifies that buildings and structures shall be founded on grounds having adequate supporting capacity. This statement emphasizes that nuclear power plant structures (assume Class S) can be founded on materials other than rock provided the material has adequate supporting capacity. Seismic (base) isolation is also considered as an option.

5.3 Japanese and U.S. Structural Response and SSI Criteria

U.S. and Japanese (JEAG 4601-1987) approaches are very similar in treating SSI and structure modeling and analysis with the following exceptions:

- Soil profiles (United States more conservative)
    - U.S. practice is to analyze three soil profiles to account for uncertainty in soil behavior and behavior of the soil-structure system and then envelope results
    - Japan practice is to analyze one soil case (based on review of JEAG 4601-1987, it appears to be the best estimate soil profile, which accounts for strain-dependent soil properties)
- Structure damping (Japan more conservative)
    - Japan damping values are less (in some cases much less) than those specified in NRC RG 1.61
- Nonlinear behavior of the soil-structure interface (generally equivalent in its impact on the seismic design)
    - U.S. practice is to perform a check to determine if sliding or uplift occurs and embedment soil separation is treated by equivalent linear methods
    - Japan practice is to treat sliding and uplift explicitly for the S_2 earthquake ground motion
- Nonlinear behavior of structure (Japan more realistic)
    - U.S. practice treats the structure as equivalent linear
    - Japan practice analyzes Class A_s buildings/structures for nonlinear structure behavior under the S_2 earthquake ground motion

6 STRUCTURAL DESIGN APPROACHES

The discussion of the structure seismic design process in both countries is separated into three parts: (1) structure load determination, (2) structure design considerations, including load combinations, and (3) a discussion of key similarities and differences between criteria in the United States and Japan. The comparison is difficult and best performed through a pilot study as described in Section 10.

6.1 Structure Seismic Loads

6.1.1 U.S. Criteria (Pre-2007 and Post-2007)

The key elements of seismic load determination for the U.S. nuclear power plants (NPPs) were introduced in Sections 3.1 and 5.1. Those elements important to this discussion are:

- Seismic loads for the Operating Basis Earthquake ground motion and Safe Shutdown Earthquake ground motion are determined from dynamic analysis of the soil-structure system considering three components of ground motion simultaneously (two horizontal and the vertical).
- Except for the case of hard rock, three soil profiles are considered in the analysis: best estimate, lower bound, and upper bound. Seismic design loads are taken as the envelope of the responses for the three soil cases.

6.1.2 Japan Criteria (Pre-2006)

In Pre-2006 designs, the Japan Electric Association (JEA) guidance document JEAG 4601-1987 separates the seismic design loading requirements by Class of structure. For Class A_s and A structures, the structure loads for design are the envelope of the dynamically calculated responses due to the S_1 earthquake ground motion and the statically calculated values for the static coefficient listed in Table 6-1. An additional requirement is that the Class A_s structures remain functional when subjected to the S_2 earthquake ground motion. For Classes B and C structures, the structure loads are statically calculated values.

The horizontal plus vertical analyses are assumed to be applied separately for each of the two horizontal directions and the maximum used for structure design.

For the static analyses, a reduction in the horizontal force for portions of the structure below plant grade is permitted.

6.1.3 Japan Criteria (Post-2006)

As discussed in Sections 3.2 and 4.2, the Post-2006 Japanese criteria Nuclear Safety Commission (NSC) Regulatory Guide (RG) combines Classes A_s and A into Class S and defines a new set of design-basis ground motions (S_s and S_d). In general, Class S structures, systems and components (SSCs) should be designed to remain elastic when subjected to the maximum of the static calculated values and the dynamically calculated responses due to the S_d earthquake ground motion. Class S SSCs are further evaluated to the S_s with the requirement that these SSCs remain functional. Table 6-2, below, summarizes the requirements for seismic forces in NPP buildings.

Table 6-1 Summary of the requirements for seismic forces in NPP buildings after JEAG 4601-1987

Class	Static Analysis	Dynamic Seismic Force	
		S_1 ground motion	S_2 ground motion
A_s	▪ Horizontal seismic force calculated from $3xC_l$ ▪ Vertical seismic force calculated from C_v	▪ The horizontal seismic force is the seismic force on the building due to the S_1 ground motion ▪ The vertical seismic force is calculated by taking half of the maximum acceleration amplitude of the S_1 ground motion as the vertical seismic coefficient*	▪ The horizontal seismic force is the seismic force on the building due to S_2 ground motion ▪ The vertical seismic force is calculated by taking half of the maximum horizontal acceleration amplitude of the S_1 ground motion as the vertical seismic coefficient*
A	▪ Horizontal seismic force is calculated from $3C_l$ ▪ Vertical seismic force is calculated from C_v	▪ The horizontal seismic force is the seismic force on the building due to S_1 ground motion ▪ The vertical seismic force is calculated by taking half of the maximum horizontal acceleration amplitude of the basic earthquake ground motion as the vertical seismic coefficient	
B	▪ Horizontal seismic force is calculated from $1.5C_l$	Not taken into consideration (investigation is conducted for equipment and piping with the possibility of resonance)	
C	▪ Horizontal seismic force is calculated from C_l		

Notes:

C_l (story shear coefficient): Value determined using 0.2 as the basic shear coefficient and with consideration of the dynamic characteristics of the structure, type of ground, etc.

C_v (vertical seismic coefficient): Value determined using 0.3 as the basic value of the coefficient, and with consideration of the dynamic characteristics of the structure, type of ground, etc.

* Both horizontal seismic force and vertical seismic force take place simultaneously combined in unfavorable directions. The vertical seismic force is considered to be constant in the vertical direction.

Table 6-2 Seismic forces for NPP buildings (NSC Regulatory Guide 2006)

Class	Static Analysis	Dynamic Seismic Force	
		S_d Ground Motion	S_s Ground Motion
S	▪ Horizontal seismic force calculated from 3 C_I ▪ Vertical seismic force calculated from C_v	The seismic forces are the result of dynamic analyses including the effects of horizontal and vertical ground motion simultaneously	The seismic forces are the result of dynamic analyses including the effects of horizontal and vertical ground motion simultaneously
B and C	Same as JEAG 4601-1987(1)	Same as JEAG 4601-1987(1)	Same as JEAG 4601-1987(1)

Notes:

C_I (story shear coefficient): Value of 0.2 used as the basic shear coefficient and by taking into consideration the dynamic characteristics of the structure, type of ground, etc.

C_v (vertical seismic coefficient): Value of 0.3 used as the basic value, and by taking into consideration the dynamic characteristics of the structure, type of ground, etc.

(1) See Table 6-1 of this document

The dynamic analyses are assumed to be two sets of analyses, one for each horizontal direction and with each analysis including the vertical ground motion. The maxima are then used for structure design or evaluation.

For the static analyses, additional considerations apply:

- The seismic story shear coefficient C_I is further defined as a function of the seismic zone in which the plant is located, the vibration characteristics of the structure, a parameter related to vertical force distribution, and the standard shear coefficient (0.2g). The effect of these parameters on the standard shear coefficient has not been determined for a representative plant.
- A reduction in the horizontal force for portions of the structure below plant grade is permitted. Significant detail is provided in NSC RG (2006) as to how to modify the static coefficients for below grade portions of the structure. This concept is not new. It was included in JEAG 4601-1991 and the same practice is contained in the Building Standard Law in Japan.

6.2 Structure Seismic Design

6.2.1 U.S. Practice

Nuclear Regulatory Commission (NRC) Standard Review Plan (SRP) Sections 3.8.1 through 3.8.4 provide acceptable approaches to the design of structures - concrete containment, steel containment, concrete and steel internal structures, and other seismic category I (SC-I) structures, respectively.

6.2.2 SRP Section 3.8.1 Concrete Containment, Revision 2, March 2007

SRP Section 3.8.1 specifies overall acceptable design procedures for concrete containments which include references to RGs and Industry Codes for additional details. Two particularly relevant documents are:

- American Society of Mechanical Engineers (ASME), Boiler and Pressure Vessel Code, Section III, “Code for Concrete Reactor Vessels and Containments,” Division 2.
- RG 1.136, “Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments,” Revision 3, March 2007.

6.2.3 SRP Section 3.8.2 Steel Containment, Revision 2, March 2007

SRP Section 3.8.2 specifies overall acceptable design procedures for steel containments and steel components of concrete containments that resist pressure and are not backed by structural concrete. RGs and Industry Codes are referenced for details of the design process. Two particularly relevant documents are:

- ASME, Boiler and Pressure Vessel Code, Section III, Division 1, Subsection NE, “Class MC Components,” 2001 Edition with 2003 Addenda.
- RG 1.57, “Design Limits and Loading Combinations for Metal Primary Reactor Containment System Component,” Revision 1, March 2007.

6.2.4 SRP Section 3.8.3 Concrete and Steel Internal Structures of Steel or Concrete Containments, Revision 2, March 2007

SRP Section 3.8.3 specifies overall acceptable design procedures for the design of internal structures that support and protect the nuclear supply steam system (NSSS) and peripherals. In addition to the ASME Codes cited above, other particularly relevant documents are:

- American Concrete Institute, “Code Requirements for Nuclear Safety Related Concrete Structures (and its supplements),” ACI 349-97.
- American National Standards Institute/American Institute of Steel Construction, “Specification for Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities,” ANSI/AISC N690-1994 including supplement 2 (2004).
- RG 1.142, “Safety-Related Concrete Structures for Nuclear Power Plants (other than Reactor Vessels and Containments),” Revision 2, November 2001.
- RG 1.199, “Anchoring Components and Structural Supports in Concrete,” November 2003.

6.2.5 SRP Section 3.8.4 Other Seismic Category I Structures, Revision 2, March 2007

SRP Section 3.8.4 specifies overall acceptable design procedures for the design of SC-I structures other than those internal structures that support and protect the NSSS and peripherals. Generally, the RGs, Industry Codes, and ASME Codes cited above apply.

SRP Sections 3.8.1 through 3.8.4 specify loads and load combinations directly or with reference to the documents cited above. Appendix A to SRP Section 3.8.4 specifies requirements for reinforced masonry walls in safety-related structures. Un-reinforced masonry is prohibited for new design. Guidance is also provided in SRP Section 3.8.4 for other types of structures, such as retaining walls and water control structures that could have an effect on plant safety. This guidance is by reference to industrial standards.

Other structures that are not classified as safety related are designed and constructed to conventional building standards such as the National Building Code with reference to standard ASCE 7, the Uniform Building Code, or other building codes that are specified by local building authorities. These codes are designed to prevent collapse and loss of life for a specified earthquake ground motion that may be lower than the safe shutdown earthquake (SSE) ground motion for safety-related structural design.

Other structures that are safety-related, but may not necessarily be classified as SC-I, are to be designed for the SSE. SRP Section 3.2.1.3 requires that "those SSCs not identified as SC-I, but whose failure could reduce the function of any SC-I feature to an unacceptable safety level or result in incapacitating injury to control room personnel, have been identified for analysis to assure they will not fail during an SSE."

Generally, in terms of seismic loading conditions, the significant issue of load combination important to this study is the inclusion of loss-of-coolant accident (LOCA) loads in combination with seismic loading conditions. LOCA loads directly applied or their indirect effect, e.g., induced vibrations, are to be combined with seismic loads, the operating basis earthquake (OBE) ground motion (if applicable) and the SSE.

6.2.6 Japan Practice (Pre-2006)

For Pre-2006, the guideline JEAG 4601-1987 provides overall guidelines for acceptance criteria and load combinations. For Classes A_s and A SSCs, Table 6-3 summarizes these criteria where the nomenclature includes the following:

- Normal loads (e.g., dead load, live load, earth pressure, water pressure)
- Operating loads
- Accident loads
- Earthquake and other environmental loads (Earthquake ground motion loads are S_1* and S_2, where S_1* is the envelope of the S_1 dynamic response and the static loads)

The loads induced by earthquake motions and by other dynamic events need not be combined by absolute sum, if it can be shown that the peaks of the loads do not overlap in time.

6.2.7 Japan Practice (Post-2006)

The NSC RG (2006) provides overall guidance on the design requirements for Class S structures.

- Envelope of seismic design forces and moments for the S_d earthquake ground motion and the static analysis results. Load combinations (S_d* + Normal loads + Operating loads) and allowable limits parallel those of the S_1* for the Pre-2006 criteria.
- The seismic design forces/moments for the S_s are treated in combination with the normal loads and operating loads as for the S_2 criteria for the Pre-2006 criteria. The acceptance criteria are that there should be adequate safety margin when compared to ultimate strength of buildings and structures.

For Classes B and C structures, allowable stresses based on a suitable standard building code applies. Table 6-4 itemizes the load combinations and allowable limits for the Post-2006 criteria.

Table 6-3 Load combination and allowable limits (JEAG 4601-1987)

Class	Structure	
	Load Combinations	Allowable Limits
A_s	S_1* + Normal loads + Operating loads + Accident Loads(1) and S_2 + Normal loads + Operating loads	Should have safety margin with respect to the ultimate strength capacity
A_s and A	S_1* + Normal loads + Operating loads	Short term allowable stress based on a suitable Standard Building Code
B	Static seismic force for Class B + Normal loads + Operating loads	Same as above
C	Static seismic force for Class C + Normal loads + Operating loads	Same as above

Notes:
(1) Even for a phenomenon that may not be caused by earthquake, if the phenomenon lasts over the long period of time when an accident takes place, the load due to this phenomenon should be combined with the S_1*. The requirement for the combination is a function of the duration of the independent accident event and the probability of its occurrence. In all cases, operating loads are dependent on the state of the facility.
S_1* - Envelope of S_1 dynamic loads and static loads

Table 6-4 Load combination and allowable limits (JEAC 4601-2008)

Class	Buildings and Structures	
	Load Combinations	Allowable Limits
S	S_s + Normal loads + Operating loads	Should have safety margin with respect to the ultimate strength capacity
S	S_d* + Normal loads + Operating loads	Short term allowable stress based on a suitable Standard Building Code
B	Static seismic force for Class B ($1.5C_I$) + Normal loads + Operating loads	Same as above
C	Static seismic force for Class C ($1.0C_I$) + Normal loads + Operating loads	Same as above

Notes:
S_d* - Envelope of S_d dynamic loads and static loads ($3C_I$)

6.3 U.S. and Japan Structural Design Considerations

Seismic Loads

Areas where the U.S. practice is more conservative than Japanese practice include:

- Three components of ground motion considered to act simultaneously
- Envelope of three soil cases used

Areas where Japanese practice is more conservative than U.S. practice include:

- Structure damping values

Structure Design

Some details of the allowable stress and deformation limits are provided in Chapter 5 of JEAG 4601-1987. However, a meaningful discussion of the similarities and differences between these criteria with that of the United States is best performed by selecting a number of specific situations and comparing the loads to be considered, load combinations to be considered, classification of the structures, and the design details. Initial effort in this regard is the Kashiwazaki-Kariwa Research Initiative for Seismic Margin Assessment (KARISMA) benchmark project that was conducted as part of an International Atomic Energy Agency Extra Budgetary project. Additional benchmark studies could be extremely helpful. If such studies were to be performed, a selection of samples to be studied should range over the typical situations encountered in the design of pressurized-water reactor and boiling-water reactor configurations.

One difference that is apparent is the U.S. requirement to consider the combined loadings of seismic and LOCA for design. LOCA loads used in design may be directly or indirectly induced and they are influenced by elements such as leak before break provisions. The Japanese practice is that the S_1 earthquake loads should be combined with accident loads for long duration accident states. The criteria for combining of S_1 seismic loads and loads resulting from accident states depends on the probability of occurrence of the accident and its duration. The U.S. practice should generally lead to more conservatism.

7 SUBSYSTEM ANALYSIS AND DESIGN AND EVALUATION METHODS

7.1 General

Generally, the U.S. practice of seismic analysis and design and qualification of subsystems is consistent from the most recent operating plants (vintage 1980s to 1990s) to new plants currently being designed, the majority of which are Certified Designs. Differences are in the lack of explicit design for the operating basis earthquake (OBE) ground motion (OBE $\leq$ 0.33 x SSE (Safe Shutdown Earthquake) for new plants) [1]. The discussion in this section is organized according to type of subsystem and the U.S. and Japan practice for existing plants is discussed for each. The following references are applicable to the U.S. practice:

- NUREG-0800 (Formerly NUREG-75-087), "Standard Review Plan (SRP)," Office of Nuclear Reactor Regulation
- Regulatory Guide (RG) 1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants," Revision 1, 1973
- RG 1.61, "Damping Values for Seismic Design of Nuclear Power Plants," Revision 1, March 2007
- RG 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," Revision 2, 2006
- ASCE/SEI 43-05, "Seismic Design for Structures, Systems and Components in Nuclear Facilities," American Society of Civil Engineers/Structural Engineering Institute, 2005
- ASCE 4-98, "Seismic Analysis of Safety-Related Nuclear Structures and Commentary," American Society of Civil Engineers, 2000 (in revision 2010)
- AISC, "Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings," 8th Edition, American Institute of Steel Construction, 1980
- ASME Boiler and Pressure Vessel Code, Section III, "Rules for Construction of Nuclear Power Plant Components," Division I, American Society of Mechanical Engineers, 2004 (including Addenda)
- IEEE Standard 344-2004, "IEEE Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations," Institute of Electrical and Electronics Engineers, Inc., 2004
- IEEE Standard 323-2003, "Standard for Qualifying Class 1E Equipment for Nuclear Power Generating Stations," Institute of Electrical and Electronics Engineers, Inc., 2003
- NUREG/CR-3660, "Probability of Pipe Failure in the Reactor Coolant Loops of Westinghouse Pressurized Water Reactor (PWR) Plants," Volumes 1-4, July 1985
- NUREG/CR-4792, "Probability of Failure in Boiling Water Reactor (BWR) Reactor Coolant Piping," Volumes 1-4, December 1988

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[1] SSE is the safe shutdown earthquake ground motion

7.2 Generating Seismic Response for Subsystem Design and Qualification

Section 5 discussed the soil-structure interaction (SSI) and structure response portion of the process. For analysis of decoupled subsystems, in-structure response spectra (ISRS) define the input motion. Additionally, relative motions at various locations in the structures are available to analyze multi-supported subsystems.

7.2.1 U.S. Approach Pre-2007

Before 2007, Seismic Category I (SC-I) components were required to be designed for the OBE and SSE ground motions using either dynamic response analysis procedures or a conservative static coefficient method. Dynamic analysis, if performed, uses ISRS or earthquake acceleration time series as input motion. If in-structure time histories from the SSI and structure response analyses are used, procedures are required to account for the uncertainty in these input motions. These procedures are required to address enveloping, smoothing, peak broadening, and filling in valleys with equivalent approaches to those used for the ISRS. For the conservative static coefficient approach, the static coefficient is defined as 1.5 times the peak spectral acceleration of the ISRS at the appropriate damping values for the equipment. If the component can be shown to be rigid, the zero period acceleration can be used as the static coefficient. Also, it is common practice for the specifications for equipment, such as pumps and valves, to require a design for a static coefficient ranging from 3g up to 6g regardless of the ISRS demand. Post-2007, the OBE design requirement has been removed when the OBE $\leq$ 0.33 x SSE for new nuclear power plants (NPPs).

7.2.2 Japan Approach Pre-2006

The general philosophy in the Japan Electric Association (JEA) guidance document JEAG 4601-1987 is that the components and subsystems should be stiff. The term "stiff" is defined as the first natural frequency of the component and subsystem is higher than the dominant frequency range of the input ISRS. It is assumed that the component or subsystem is "stiff" if the first natural frequency is greater than 20 Hz. If components and subsystems are stiff, their response to earthquake ground motions would be reasonably low, especially if the components or subsystems are mounted low in the structure. The guideline JEAG 4601-1987 requires that both a static design and dynamic design analysis be conducted for Class A_s and A components.

- Class A_s components are to be designed or evaluated for three different seismic demands. The basic design is for the dynamic response to the S_1 earthquake ground motion with an alternate static design analysis using horizontal and vertical static coefficients. In addition, the dynamic response for the S_2 earthquake ground motion is to be evaluated relative to higher allowable stresses to assure that excessive deformation does not occur or that function would not be substantially degraded.
- For class A components, the dynamic response to the S_1 earthquake ground motion is calculated and compared to allowable stresses and the alternate static design analysis is performed. The resulting seismic design is governed by the more severe of the dynamic or static demand.
- Class B components are designed using static analysis equal to ½ of the static analysis requirements for class A components. An exception is, if the fundamental frequency is in the highly amplified portion of the ISRS, the dynamic response to ½ of the S_1 earthquake ground motion is used for comparison to the static coefficient.

Table 7-1 compares the U.S. seismic demand for the OBE and SSE for SC-I equipment with the JEAG 4601-1987 requirements for Classes A_s, A, and B.

Table 7-1 U.S. and JEAG 4601 subsystem design response parameters

Seismic Class	Static Analysis		Dynamic Analysis	
	Horizontal	Vertical	Horizontal	Vertical
U.S. Category 1	1.5 (Peak S_a)	1.5 (Peak S_a)	OBE & SSE ISRS	OBE & SSE ISRS
JEAG Class A_s			S_2 ISRS	½ of Peak S_2 Ground Motion
JEAG Class A, A_s	3.6C_I	1.2C_V	S_1 ISRS	½ of Peak S_1 Ground Motion
JEAG Class B	1.8C_I	Not specified	½ of S_1 ISRS only if component in resonance with structure	Not specified
Notes: The static coefficients C_I and C_V in the above table are 0.2g and 0.3g respectively. S_a is spectral acceleration				

If equipment is stiff, the static coefficient design for Class A components may govern the design depending upon the response of the structure housing the equipment. Use of 1.5 times the peak of the response spectrum for U.S. static design is always conservative and is usually avoided unless the equipment is compact and inherently rugged and it is easy to show that resulting stresses are low.

The dynamic analysis methods employed in U.S. and Japanese design are very similar. The notable variations are in the damping values and the method of combining the three directions of earthquake ground-shaking components.

7.2.3 Damping Values in U.S. and Japan Practice

Damping values used in U.S. designs are specified in U.S. Nuclear Regulatory Commission (NRC) RG 1.61, "Damping Values for Seismic Design of Nuclear Power Plants." RG 1.61 as it was originally published did not provide guidance for damping of mechanical and electrical equipment, cable raceways and heating, ventilation, and air conditioning (HVAC) ducting. Revision 1 to RG 1.61 includes damping values for these components and subsystems. Existing designs employed various damping values for these components and subsystems, which were accepted by the NRC during the licensing process. Some early designs used very conservative damping values relative to the values that were contained in the initial issuance of RG 1.61 in 1973.

In the time frame of the 1987 publication of JEAG 4601, the Japanese damping values were generally more conservative than the U.S. damping values. Table 7-2 compares damping values of U.S. and Japanese practice in 1987 and damping values currently specified in RG 1.61.

Table 7-2 Damping values used in operating U.S. plant designs, new U.S. plant designs, and JEAG 4601-1987

Equipment	Pre 2007 U.S. (SSE)	Current U.S. (SSE)	JAEG 1987 Class A and As Piping (S_1 and S_2 Earthquake)
Piping	2% ≤ 30 cm (12 in) 3% > 30 cm (12 in) Variable 5% to 2%*	4% (all diameters) Variable 5% to 2%*	0.5% to 2.5%
Mechanical Equipment	2% to 3% **	3%	1%
Electrical Equipment	2% to 3% **	3%	4%
Primary Coolant System (PWR)	4% based on NSSS supplier testing	4%	3%
Equipment Supports	2% to 3%**	3%	2%
Welded Steel Supports	4%	4%	1%
Cable Trays	Case by case**	7% to 10%	5%
Ducting	Case by case**	4% to 10%	2.5%
Fluid Sloshing	0.5%	0.5%	0.5%

Note: Current U.S. damping values are from values in RG 1.61, rev. 1

* Variable damping of 5 percent up to 10 Hz and varying linearly to 2 percent at 20 Hz may be used with envelope response spectrum analysis and RG 1.60 ground motion input spectrum.

** Values were not specified in RG 1.61. Damping values were agreed upon in the licensing process. In general, equipment damping was equivalent to piping damping. Damping for cable trays and HVAC ducting were negotiated with regulators depending upon time frame and particular design details since different design details warranted different damping values.

The variation in piping damping in JEAG 4601-1987 depends on how the piping is supported and if it is insulated. Table 7-3 summarizes the conditions associated with the different damping values. In some cases, the JEAG 4601-1987 damping was more liberal than RG 1.61 but in most cases, it is more conservative. It should be noted that damping in the original version of RG 1.61 for the OBE ground motion was lower and in the case of piping, often the OBE ground motion governed the design as a result of the high response to low damping and the lower allowable stress for OBE load combinations which was ½ of the stress allowed for SSE load combinations. The introduction of the variable damping from 5 percent to 2 percent for piping systems was applicable to both OBE and SSE response and eliminated the situation where OBE ground motion governed piping design. After the introduction of the variable damping, some U.S. utilities reevaluated their piping using the higher variable damping and were able to remove snubbers and avoid the costly inspection and maintenance associated with installed snubbers.

Table 7-3 Damping values for piping in JEAG 4601-1987 for class A and A_s piping

Piping Type		Damping Constant for Design (%)	
		With Insulation	Without Insulation
I	Piping systems supported mainly by frame restraints and snubbers with four or more supports	2.5	2.0
II	Piping systems with snubbers, frame restraints, rod hangers, etc. with four or more supports (excluding anchors and U-bolts), and not belonging to Type I	1.5	1.0
III	Piping systems not belonging to Piping Type I or II	1.0	0.5

Damping values have been revised in JAEC 4601-2008 and include damping for the vertical direction to reflect that dynamic analysis is now required for the vertical direction. Tables 7-4 and 7-5 show the current damping values for equipment and piping, respectively.

7.2.4 Earthquake Components of Motion Used in Design

Pre- and Post-2007, the U.S. practice is to consider three components of earthquake input motion (two horizontal and the vertical) for design and qualification. The U.S. practice for combination of responses due to each of the three earthquake components is specified in RG 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis." The latest revision provides for the three components of response due to the three earthquake input motions to be combined by square root sum of the squares (SRSS) or by absolute sum of 100-40-40 combination of response to the three earthquake input motions. In this latter case, the governing case of the 100-40-40 combination is used. In general, the response results are comparable. The advantage of the 100-40-40 combination is that the combination can be applied to the load components or the resulting stresses whereas, in the SRSS combination, the end item of interest is to be combined by SRSS. Often, designers had inappropriately applied the SRSS combination to the load components, thus inadvertently calculating a vector of the three components in phase. Use of the 100-40-40 rule is less confusing. Prior to the issuance of RG 1.92 in 1974, early U.S. practice was to combine the worst horizontal response with the vertical response by absolute sum. This combination can, however, be un-conservative.

The guidelines in JEAG 4601-1987 require the response of the dynamic horizontal component to be combined with a vertical static coefficient by absolute sum. It is assumed that the requirement is to base the design on the worst combination of one horizontal component with the vertical static coefficient. The vertical static coefficient is based on ½ of the peak horizontal ground motion. If the plant components are stiff and the structure is stiff in the vertical direction, the static coefficient appears reasonable, but if a flexible plant component is on a flexible floor, the actual vertical dynamic response would be significantly greater than predicted by the static coefficient. In general, the vertical component of earthquake has little influence on the seismic design but in some cases the vertical component response can be high and influence the design. JEAG 4601-1987 refers to research that shows no significant effect of the method of earthquake component combination, but details are not provided.

JAEG 4601-2008 provides the following guidance for combination of earthquake components.

When dynamic analysis is conducted for three components of earthquake motion, the combination of responses may be by SRSS or the 100-40-40 rule or, in the case of time history analysis, the combination is by algebraic sum at each time step. When static analysis is conducted, the combination is by absolute sum of horizontal and vertical components.

Table 7-4 Damping values for equipment in JAEC 4601-2008

Equipment	Damping Constant for Design (%)	
	Horizontal	Vertical
Reactor Fuel Assembly (PWR)	-[(1)]	1.0
Reactor Fuel Assembly (BWR)	7.0	1.0
Control Rod Drive Mechanism (PWR)	5.0	1.0
Control Rod Drive Mechanism (BWR)	3.5	1.0
Air Ducting (Welded Steel with Rectangular or Circular Section)	2.5	2.5
Cable Tray (Steel Solid Type or Ladder Type)	5.0	5.0
Self-standing Closed Type Electrical Panel	4.0	1.0
Crane (Overhead and Refueling)	2.0	1.5-2.0
Primary Cooling System (PWR, Steam Generator, Reactor Pump and Piping)	3.0	1.0
Extracting Tube for Core Monitor (PWR)	2.5	2.5
Steam Generator Heat Transfer Tube (PWR)	8.0 (out of plane) 15.0 (in plane)	1.0
Spent Fuel Storage Rack (PWR, Angle type cell)	5.0 ($f<20Hz$) 3.0 ($20Hz \leq f<30Hz$)	1.0
Spent Fuel Storage Rack (PWR, Can type cell)	7.0 (f[(2)]<20Hz) 5.0 ($20Hz \leq f<30Hz$)	1.0
Mechanical Equipment	1.0	1.0
Welded Steel Structures	1.0	1.0
Bolted or Riveted Steel Structures	2.0	2.0

Notes:

(1) Displacement dependent (e.g., equivalent damping value at 10mm is 10 to 15 percent)
(2) First natural frequency

Table 7-5 Damping values for piping in JAEC 4601-2008

Piping Type		Damping Constant for Design (%)	
		With Insulation	Without Insulation
I	Piping system supported mainly by frame restraints and snubbers with four or more supports	3.0	2.0
II	Piping systems with snubbers, frame restraints and rod hangers, etc. with four or more supports (excluding anchors and U-bolts), and not belonging to Type I	2.0	1.0
III	Piping systems supported with four or more U-bolts set on the frame structure bearing the dead weight of horizontal piping	3.0	2.0
IV	Not belonging to Piping Type I, II, or III	1.5	0.5

7.3 Subsystem Design and Qualification

The ASME Boiler and Pressure Vessel code is utilized in the United States as well as in many other countries. Some exceptions are applied by NRC in RG 1.193, "Availability of Electric Power Sources," for code cases. Acceptance and non-acceptance of specific parts of codes and standards are listed in Title 10 of the *Code of Federal Regulations* (10 CFR) 50.55a, "Codes and Standards." Appendix N of the ASME code for dynamic analysis is overridden by RGs in cases where RGs and other regulatory positions are more conservative than the criteria in Appendix N. In 10 CFR 50.55a(b)(1) (iii), seismic design of piping, the NRC did not accept the ASME code rules in NB-3200, NB-3600, NC-3600 and ND-3600 beyond the 1993 addenda. This had been a controversial issue for a number of years and industry felt strongly that the 2001 edition of the code and beyond more realistically addressed the inherent margin in reversing dynamic loads, such as produced by earthquakes. The NRC maintained its position that the version of Section III of the code for seismic design of piping beyond the 1993 addenda did not adequately address the issues identified in NUREG/CR-5361 [NRC 1998] and determined that they were unacceptable. Regulations in 10 CFR 55a(b)(1)(iii) now accept the ASME code Subarticles NB-3200 from the 2004 edition through the 2008 addenda and NB-3600, NC-3600, and ND-3600 for the seismic design of piping with minor changes.

Qualification of equipment by testing is covered by Institute of Electrical and Electronics Engineers (IEEE) standard 344. This standard is accepted by the NRC in RG 1.100, "Seismic Qualification of Electric and Mechanical Equipment for Nuclear Power Plants," except the provision for qualification by seismic experience. While seismic experience is not flatly rejected, the applicant is required to demonstrate the applicability, which would likely not be possible in light of the guidance in SECY-93-087 [NRC 1993] that the high confidence of low probability of failure (known as HCLPF) value be 1.67 times the SSE.

The U.S. practice for design has been to conduct linear elastic analysis for the OBE and SSE and compare the resulting response to allowable stresses that are typically less than yield for the OBE and beyond yield for the SSE. The ASME code has provisions for limit analysis and plastic instability analysis with appropriate response limits but such analyses are rarely done. Only primary stresses are checked for the SSE. For the OBE, primary stresses are checked for ASME Class 1, 2, and 3 components and piping and secondary stress checks and fatigue analysis are conducted for ASME Class 1 components and piping. Class 2 and 3 components and piping do not require secondary stress checks and fatigue analysis since they typically have few extreme cycles of temperature and mechanical load.

The philosophy of JEAG 4601-1987 is that the equipment should remain essentially elastic (stress state III_AS) for the S_1 earthquake ground motion but for the S_2 earthquake ground motion, non-linear analysis is encouraged and the allowable stresses (stress state IV_AS) are beyond yield. Also, for Class A_s and Class A equipment subjected to the S_1 and S_2 earthquake ground motions, secondary stresses are to be evaluated and fatigue analysis is required.

Following is a discussion of the similarities and differences between U.S. and JEAG 4601-1987 design requirements for vessels, piping, component supports and for electrical and Instrumentation and Control (I&C) equipment.

7.3.1 Pressure Vessel Design Criteria

Pressure vessel seismic design criteria in ASME codes and JEAG 4601 are very similar. JEAG Type 1 components are those that constitute the primary coolant system. The analogous classification in ASME is Class 1 components. JEAG Type 2 vessels are steel containment, which is analogous to ASME Subsection MC for steel containment. JEAG Type 3 components are safety-related components that are not Type 1 or 2 and are analogous to ASME Class 2 and 3 components.

Table 7-6 shows the stress checks required for stress classes III_AS and IV_AS for Type 1 and Type 3 vessels (equivalent to ASME Class 1 and Classes 2 and 3). Stress state III_AS allowable stresses must generally be met for the S_1 ground motion and stress state IV_AS must be met for the S_2 ground motion.

Note that the requirement for secondary stress analysis and fatigue analysis for stress state IV_AS for Type 1 components would employ the S_2 ground motion (equivalent to SSE). Because the range of seismic stress (plus to minus) is evaluated for secondary and peak stresses as opposed to the maximum stress for primary stress evaluation, the secondary and peak stresses could govern if evaluated for the S_2 earthquake ground motion. Evaluation of secondary and peak stresses for the S_2 ground motion appears to be a mistranslation of JEAG 4601 as it would not be in keeping with the basic design philosophy of designing for S_1 ground motions and evaluating for S_2 ground motions to assure that excessive deformations do not occur and that function is not significantly degraded. The ASME code does not require evaluation of secondary stresses or fatigue analysis for the SSE ground motion on the basis that the SSE ground motion would almost certainly not occur more than once during the life of the plant and the objective is to assure that the equipment and subsystems remain functional for safe shutdown but not necessarily be able to be placed back into service after the earthquake. Per information provided by JEA, the cyclic stress for the S_S ground motion is counted in the fatigue analysis.

The JEAG 4601-1987 allowable stresses for primary membrane and primary local membrane plus primary bending for vessels appear to be very similar to the U.S. ASME code allowable stresses. Table 7-7 compares the allowable stresses for JEAG 4601 Type 1 and 3 vessels subjected to the S_2 earthquake ground motion to the ASME code allowable stresses for ASME Class 1 vessels and ASME Class 2 and 3 vessels subjected to the SSE.

Table 7-6 Allowable stresses for JEAG Type 1, Type 3, and Type 4 vessels for stress states III_AS and IV_AS

<table>
<tr><th colspan="6">Stress Class</th></tr>
<tr><th>Type of Vessel</th><th>Stress State</th><th>Primary General Membrane</th><th>Primary Membrane + Primary Bending Stress</th><th>Primary + Secondary Stress</th><th>Primary + Secondary + Peak Stress</th></tr>
<tr><td rowspan="2">Type 1</td><td>III_AS</td><td>[S_y or 2/3 S_u](1)
for ASS or HNA 1.2 S_m</td><td>1.5 times the left column</td><td rowspan="2">3 S_m (2)</td><td rowspan="2">Fatigue usage factor <1.0(3)</td></tr>
<tr><td>IV_AS</td><td>2/3 S_u
for ASS or HNA
[2/3 S_u or 2.4 S_m](1)</td><td>1.5 times the left column</td></tr>
<tr><td rowspan="2">Type 3 & 4</td><td>III_AS</td><td>[S_y or 0.6S_u](1)
for ASS or HNA 1.2 S</td><td>1.5 times the left column</td><td colspan="2" rowspan="2">Fatigue usage factor $\leq$1.0
If primary + secondary stress $\leq$ 2S_y(2), the fatigue analysis is not needed</td></tr>
<tr><td>IV_AS</td><td>0.6 S_u</td><td>1.5 times the left column</td></tr>
<tr><td colspan="6">Notes:
1 The lesser value is applicable
2 Evaluation is made for seismic stress range only
3 Fatigue usage factor from seismic motion only. To be added to usage factor from operating states I and II
S_u=Ultimate tensile strength
S_y=Yield strength
ASS=Austenitic stainless steel
HNA=High nickel alloy
S_m=Allowable stress for Type 1 components (equivalent to ASME Class 1)
S =Allowable stress for Type 3 components (equivalent to ASME Classes 2 & 3)</td></tr>
</table>

Table 7-7 Allowable stresses for carbon steel vessels from JEAG 4601-1987 and ASME

Type of Stress	JEAG Type 1	ASME Class 1	JEAG Type 3	ASME Class 2 and 3
Primary membrane	2/3 S_{ult}	0.7 S_{ult}	0.6 S_{ult}	2 S*
Primary local membrane + primary bending	S_{ult}	1.05 S_{ult}	0.9 S_{ult}	2.4 S*

Notes:

*S is the allowable stress for ASME Class 2 and 3 vessels defined as the lesser of 5/8 of yield and 1/4 of ultimate. Later ASME code stress tables define the allowable stress, S, as the lesser of 2/3 of yield or the ultimate strength divided by 3.5. Most existing plants were designed using the earlier version of S.

Stress limits for austenitic stainless steel vessels and high nickel alloy vessels are slightly different due to the lower ratio of yield stress to ultimate strength, but the comparisons in Table 7-7 for carbon steel are approximately applicable to austenitic stainless steel vessels and high nickel alloy vessels.

The guidelines in JEAG 4601-1987 are slightly more conservative for Type 1 vessels and slightly less conservative for Type 3 Vessels. The allowable stress criteria are considered to be comparable.

JAEC 4601-2008 has changed the stress limit designations from III_AS and IV_AS to C_S and D_S. Table 7-8 provides the JEAC 4601-2008 limits.

Table 7-8 Allowable stresses for JEAC Type 1, Type 3, and Type 4 vessels for stress states C_S and D_S

Stress Class					
Type of Vessel	Stress State	Primary General Membrane	Primary Membrane + Primary Bending Stress	Primary + Secondary Stress Range	Primary + Secondary + Peak Stress
Type 1	C_s	[S_v or 2/3 S_u](1) for ASS or HNA 1.2 S_m	α times the left column	3 S_m (2)	Fatigue usage factor <1.0
	D_s	2/3 S_u for ASS or HNA [2/3 S_u or 2.4 S_m](1)	α times the left column		
Type 3 & 4	C_s	[S_v or 0.6S_u](1) for ASS or HNA 1.2 S	1.5 times the left column	Fatigue usage factor ≤1.0 If primary + secondary stress ≤ 2S_v(2), the fatigue analysis is not needed	
	D_s	0.6 S_u	1.5 times the left column		

Notes:
(1) The lesser value is applicable
(2) Evaluation is made for seismic stress range only

7.3.2 Piping Design Requirements

Piping stress equations and allowable stresses are comparable. Table 7-9 compares piping stress equations in JEAG 4601-1987 to those in the ASME code.

Table 7-9 Stress equations in JEAG 4601-1987 and ASME code

Design Standard	Piping Stress Equation
JEAG Type 1	$\frac{B_1 PDo}{200t} + \frac{B_2 M_{ip}}{Z_i}$
ASME Class 1	$\frac{B_1 PDo}{2t} + B_2 D_o \frac{M_A + M_B}{2I}$
JEAG Type 3	$\frac{PDo}{400t} + 0.75i \frac{M_A M_B}{Z}$
ASME Class 2 and 3	$\frac{B_1 PDo}{2t} + B_2 D_o \frac{M_A + M_B}{2I}$
Notes: The definition of the parameters is provided in the paragraph below.	

The formulation for JEAG Type 1 and ASME Class 1 are essentially identical. The B_1 and B_2 factor are stress indices taken from the ASME code. M_{ip} in the JEAG formula is the bending moment due to mechanical load including the earthquake inertia effects whereas M_A and M_B in the ASME formula are the dead weight and seismic bending moments. Z is equal to $2I/D_o$. The difference in the 2t and 200t results from the use of kgf/cm^2 for pressure in JEAG 4601 with t in mm whereas in the ASME formula, the pressure is in psi and t is in inches. Thus, the equations for piping stress are identical.

The JEAG formula for Type 3 piping is essentially the same as the ASME code for Class 2 and 3 piping up to about the mid-1980s where a stress intensification factor (i) was used instead of a B_2 stress index. Stress intensification factors were originally developed from displacement controlled low cycle fatigue tests to simulate stress cycles resulting from restraint of thermal expansion. The 0.75i factor was applied for seismic inertia design recognizing that the number of cycles from seismic loading was significantly less than the basic value of 2000 cycles that was inherent in the development of (i) from the low cycle fatigue tests. The B_2 stress indices are based on analytical and experimental stress analysis and for elbows are about twice the value of (i) so in this respect, the ASME code, after about the mid 1980s, is more conservative than JEAG 4601-1987 for Type 3 piping. However, most ASME Class 2 and 3 piping in U.S. plants was designed using the formula shown for JEAG Type 3 piping. Very few designs for Class 2 and 3 piping were conducted using the B_2 indices. It is therefore concluded that the Japanese and U.S. design criteria for Type 3 piping and ASME Class 2 and 3 piping were equivalent.

As discussed in the first paragraph of Section 7.3, after an extensive review of dynamic tests of piping fittings, the ASME code was changed in 1994 to increase the allowable stress in piping by a factor of 1.5 for reversed dynamic loads. This was not accepted by the NRC. In the 2001 revision of the ASME code, the B_2 stress indices were reduced by a factor of 1.5 for elbows and tees and the pre 1994 stress limit was retained. This revision was still not accepted by the NRC and the *Code of Federal Regulations* restricted the design of piping to the 1993 and earlier ASME codes. The revision of the ASME code in 2006 addressed NRC concerns and the ASME code from 2006 to 2008 is now acceptable to the NRC with some slight modifications. This controversy did not affect existing nuclear plants since all piping design was completed by 1993. Thus, current

ASME code requirements for piping design that are accepted by the NRC are more liberal for cases where elbows and tees govern support placement than the ASME code in effect when JEAG 4601-1987 was developed.

The ASME and JEAG criteria for allowable stresses for piping are very similar. Table 7-10 compares allowable stresses for ASME code piping subjected to SSE load combinations to the criteria in JEAG 4601-1987 for evaluation to the S_2 earthquake ground motion.

Table 7-10 Allowable stresses in the ASME code and JEAG 4601 for load combinations with SSE ground motion and with the S_2 ground motion

Loading	JEAG Type 1 Piping (S_2)	ASME Class 1 Piping (SSE)	JEAG Type 3 Piping (S_2)	ASME Class 2 and 3 Piping (SSE)
Pressure + Bending From Dead Load and Seismic	3 S_m or 2.4 S_m if torsion > 0.73 S_m	Lesser of 3 S_m or 2 S_y	0.9 S_{ult}	Lesser of 3S or 2 S_y

It can be seen from the above table that JEAG Type 1 piping allowable stress is essentially the same with some extra conditions employed in the case of very large torsion stress in the JEAG criteria and for some cases in the ASME code for austenitic stainless steel at elevated temperature where 3 S_m can exceed 2 S_y.

In the case of JEAG Type 3 piping versus ASME Class 2 and 3, the 0.9 S_{ult} could be higher than 3S in cases where S is governed by ¼ of the ultimate strength. In general, the stress criteria are the same for the extreme loading case of S_2 or SSE earthquake ground motion.

The allowable stresses for piping in JEAC 4601-2008 are shown in Table 7-11.

Table 7-11 Allowable stresses for piping in JAEC 4601-2008

Stress Class					
Type of Vessel	Stress State	Primary Stress (Membrane + Bending)	Primary Stress (Torsion, Bending + Torsion)	Primary + Secondary Stress Range[1]	Primary + Secondary + Peak Stress
Type 1	C_s	Minimum of [2.25 S_m or 1.8 S_y]	Torsion Stress: 0.55 S_m If unsatisfied, Bending & Torsion Stress: 1.8 S_m	3 S_m	Fatigue usage factor <1.0
	D_s	When short term mechanical load other than earthquake is considered[2] min[3 S_m or 2 S_y]	Torsion Stress: 0.73 S_m If unsatisfied, Bending & Torsion Stress: 2.4 S_m		
Type 3 & 4	C_s	When short term mechanical load other than earthquake is considered:	-	2 S_y	Fatigue usage factor <1.0

		min[2.25 S_h or 1.8 S_y] When short term mechanical load other than earthquake is not considered: S_y Or 1.2 S_h for ASS or HNA			
	D_s	When short term mechanical load other than earthquake is considered$^{(2)}$: min[3 S_h or 2 S_y]	-		
Notes: (1) Only for the load due to S_s or S_d (2) Unnecessary when short term mechanical load other than earthquake is not considered					

Overall, piping design has varied significantly over the 40 years of NPP design. Theoretically, there is a significant difference in the piping stress equations used for ASME Class 2 and 3 designs. However, the seismic response analysis has tended to compensate for the differences in equations (e.g., lower damping in the time frame of using (i) instead of B_2). Also, piping has been shown by earthquake experience to be robust regardless of the design. Thus, piping designed to the varying seismic regulatory requirements that have prevailed over the last 40 years is considered to be robust in terms of its low contribution to risk.

7.3.3 Design and Qualification of Equipment Supports

JEAG 4601 has allowable stresses for support structures subjected to the S_1 and S_2 earthquakes. The ASME code has design criteria for component supports that vary depending on whether the support is linear or a plate and shell type support. Linear support criteria are patterned after the AISC code whereas plate and shell type support criteria are patterned after the ASME criteria for vessels. Supports for non-ASME components are typically designed to AISC requirements. Table 7-12 compares the JEAG allowable stress for stress states III_AS and IV_AS to the allowable stress in the ASME code for linear supports and plate and shell supports and to AISC allowable stresses for load combinations including the SSE ground motion. Note that the NRC Standard Review Plan Limits the AISC plastic design stress used for SSE load combinations to 1.6 times the working stress allowable rather than the 1.7 factor in the AISC code.

Table 7-12 Allowable stress for component supports in JEAG 4601, ASME and AISC

Loading	Member Type	JEAG Support Structure	ASME Linear Support	ASME Plate & Shell Type Support, Membrane Stress	AISC Designed Support for Non-ASME Components
Dead + Restraint of Thermal Expansion + Seismic	Structural	1.5(1.2) f(i)*	Lesser of 1.2 S_y or 0.7 S_u	Lesser of 0.7 S_u or 2.4 S_m	1.6 times working stress allowable
	Bolt tension	1.5(1.2) f(i)*	Lesser of S_y or 0.7 S_u	Lesser of S_y or 0.7S_u	1.6 times working stress allowable
	Bolt shear	1.5(1.2) f(i)*	Lesser of 0.6 S_y or 0.42 S_u	Lesser of 0.6 S_y or 0.42 S_u	1.6 times working stress allowable

Notes:

*The allowable stress $f_{(i)}$ is for tension (f_t), shear (f_s), compression (f_c), bending (f_b) and bearing (f_p) as given in [MITI 1980]. f(i) is not defined in JAEG 4601 but is believed to be similar to the AISC working stress allowable for comparable loading conditions. The MITI Technical Guidelines have been changed to [JSME 2003].

If one assumes that f(i) is the same as the AISC allowable working stress, then 1.5(1.2) = 1.8 times f(i) is higher than the 1.6 increase factor allowed by NRC on AISC allowable working stress. For linear supports, the ASME code limit for a typical structural steel like A36 would be limited by 0.7 F_{ult} = 40.6 ksi. If f(i) is 0.6 F_y as in AISC, the limiting stress would be 38.9 ksi for JEAG and 34.56 ksi for AISC, which are both less than ASME for linear supports. If the same material is used for a plate and shell type support and the loading is in membrane, the allowable would be governed by 2.4 S_m = 34.8 ksi which is lower than the allowable stress for tension in JEAG 4601 or for ASME linear supports but equivalent to 1.6 times AISC. There are many variations in the margin of component supports depending on material, failure mode, and configuration but it appears that there is no significant difference in margin between JEAG, ASME, and AISC designed component supports.

There are no changes in JEAC 4601-2008 in component support allowable stresses from JEAG 4601-1987 except that the stress categories III_AS and IV_AS have been changed to C_S and D_S, respectively.

JEAC 4601-2008 has also added criteria for seismic design methods utilizing energy absorption effects from a supporting system of mechanical equipment and piping. Typical supporting systems investigated are frame restraints with plastic deformation, vibration controlling supports, and energy absorbers such as elastic-plastic dampers, friction dampers and lead extrusion dampers for piping and lead extrusion dampers for PWR primary cooling system components.

7.3.4 Equipment Qualified by Test

For all operating reactors in the United States, equipment qualified by test is governed by the criteria of IEEE Standards 323 and 344. IEEE 323 is the master document and IEEE 344 covers seismic testing. In general, the testing must be multi-axis and multi-frequency wherein the test response spectrum (TRS) envelopes the required response spectrum (RRS) by at least 10 percent. Recent risk based recommendations in ASCE 43-05 recommend that the TRS be 40 percent greater than the RRS to assure factors of safety consistent with those for structural and equipment seismic design, thus maintaining a consistent seismic risk. JEAG 4601 has little detail on qualification by test. In both cases, a resonant search is made by slow sine sweep testing. In the case of JEAG, it merely states that sine wave or sine beat testing is to be done at the resonant frequencies observed during the resonant search. This is an acceptable method in IEEE 344 as long as it can be shown to be applicable, but is generally not accepted by the NRC. The practice in the United States in 1987 was to perform two or three axis broad frequency tests with the TRS exceeding the RRS.

The recommended practice in IEEE 344 for seismic qualification by testing is endorsed in NRC RG 1.100 with some exceptions. The NRC has not accepted the provisions for qualification by seismic experience or testing experience. RG 1.100 also accepts the provisions in ASME QME-1 [ASME 2012] for qualification of active mechanical equipment by analysis or test with the same non-acceptance of the provisions for use of seismic experience.

JEAG refers to proving tests that have been done but it is not clear from the description that their method of testing and amplitude of testing is equivalent to U.S. standards. Table 7-13 briefly compares the JEAG 4601 stated testing requirements with those of IEEE 344.

Table 7-13 Qualification of electrical and control equipment by testing

Type of Test	JEAG	US IEEE 323 and 344
Resonant Search	Sine sweep	Sine sweep
Qualification Test	Sine Wave or Sine Beat*	Multi-axis and multi-frequency TRS ≥ 1.1 RRS

Notes:

* The wording in JEAG 4601 implies that the testing is single axis, single frequency at resonant frequencies determined during the resonant search. This is theoretically acceptable for single degree of freedom systems that have no cross coupled response from different earthquake directions or as long as the testing level is significantly higher than the in-structure response spectra at the equipment location. This method of testing is generally not acceptable to the NRC. In fact, one of the key issues for resolution in the USI A-46 program was the fact that such single frequency, single axis testing had been conducted on older U.S. NPPs and the validity of such testing was not defendable.

There was a supplement to JEAG 4601-1987 issued in 1991 that enhances the description of qualification by testing. JEAC 4601-2008 has not changed the description of qualification testing. The details of the supplement to JEAG 4601 in 1991 are not specifically known, but it is described that electrical panels are qualified by analysis or test and electrical components are tested either mounted in the panels or individually. The international practice for qualification by testing generally follows the recommended practice in IEEE 344 and it is judged that the Japanese qualification by testing generally complies with IEEE 344.

At the time of this writing, as a result of the NRC 50.54(f) Request for Information Letter [NRC 2012c], the Electric Power Research Institute (EPRI), is conducting a testing program on potentially high-frequency sensitive equipment for screening and risk-assessment purposes [EPRI 2012b]. This testing program is broken into 2 phases, with the first being focused (in part) on using a wide variety of testing motions in order to investigate the benefits and drawbacks to each method. This testing program, once completed, may provide additional insights into methods for optimizing qualification testing of equipment.

7.3.5 Load Combinations

Load combinations in U.S. plants are defined in the SRP. In general, LOCA-induced transient and steady state loads following a LOCA are combined with the SSE by square-root-of-the-sum-of-the-squares (SRSS) and evaluated relative to ASME Level D Service allowable stresses. In PWRs, the LOCA load effects are principally confined to piping and equipment inside containment but in BWRs, the loads resulting from LOCA propagate through the suppression pool and into the containment structure. These hydrodynamic loads can affect piping and equipment attached to containment or equipment, such as valves, in piping systems outside of containment if the piping is attached to containment.

The JEAG 4601-1987 criteria do not combine LOCA-induced transient or steady state loads with the S_2 earthquake ground motion. The non-transient LOCA loads are included in load combinations that include the S_1 earthquake ground motion. This can result in a significant difference in the margin in the primary system because much of the margin in existing U.S. primary systems is because of the design for combined LOCA transient loading and SSE (Safe Shutdown Earthquake).

Table 7-14 compares some typical load combinations for U.S. PWRs and BWRs with load combinations specified in JEAG 4601.

The load combination of SSE and LOCA in U.S. plants is by SRSS. The addition of non-transient LOCA loads to S_1 earthquake loads in Japanese plants is assumed to be by absolute sum. Without knowing the magnitude of the S_1 and non-transient loads compared to the SRSS of SSE and LOCA transient loads, no clear decision can be made regarding the conservatism or un-conservatism of one design criteria versus the other. It seems intuitive though that U.S. policy of combining LOCA transient loads with SSE loads by SRSS would result in greater design margin.

The commentary of the Nuclear Safety Commission (NSC) RG states that: "*the loads by the possible earthquake originated events and the long-standing loads in the wake of accidents shall be combined with the seismic loads, even if the accidents are not caused directly by the earthquakes. However, the loads under accident conditions may not be necessary to consider combining with the seismic loads, if the probability of the concurrent loads is extremely low when considering the occurrence probability of this accidental event, its duration time, and the exceedance probability of the earthquake.*" The Japan Nuclear Energy Safety Organization (JNES) has provided a further description of the combination. The S_1 (S_d) earthquake loads are combined with accident condition loads depending on the probability of occurrence of the accident and the duration of the accident loading condition. Long duration accident loads of moderate probability are to be combined with the S_1 earthquake loads whereas short duration low probability accident loads are not combined. The specific requirements for these load combinations are not well understood from the description provided by JNES.

Table 7-14 Example load combinations for ASME Level D service loads and stress state IV_AS in JEAG 4601-1987

NPP Type	Load Combinations
U.S. PWR	$D + P + M + (LOCA^2 + SSE^2)^{1/2}$
U.S. BWR	$D + P + M + (HYDRO^2 + SSE^2)^{1/2}$
JEAG Class A_s	$D + P_L + M_L + S_1$
JEAG Class A_s	$D + P + M + S_2$
JEAG Class A_s	$D + P_D + M_D + S_2$

where:

D	dead load
P	normal operating pressure load
P_L	non-transient pressure load after LOCA
P_D	mechanical pressure load from operating state I or II or max design pressure load
M	mechanical load in normal operating state
M_L	non-transient mechanical load after LOCA
M_D	mechanical load in operating state I or II or mechanical design load
LOCA	transient or steady state loss-of-coolant accident load
HYDRO	transient or steady state suppression pool hydrodynamic loads

7.3.6 Most Important Similarities and Differences in U.S. and Japan Criteria and Parameters

For subsystems and components, four important seismic design parameters are more conservatively treated by U.S. criteria than by Japan criteria (JEAG 4601-1987).

7.3.6.1 Qualification by Testing

The U.S. criteria for qualification by testing emphasize multi-axis, multi-frequency testing to assure that the component input is representative of a three-dimensional earthquake input. JEAG 4601 has little detail about testing but it is implied that single axis, single frequency sine beat or sine wave testing is conducted at the resonant frequencies determined during a sine sweep resonant search. This type of testing is only valid if it can be shown that the component is in fact a single degree of freedom system sensitive to one direction of input only or, if significant over testing is conducted to envelop multi-mode, multi-direction response. This type of testing was conducted in early U.S. NPPs and became one of the critical issues to be resolved in the USI A-46 program. This method of testing is generally not acceptable to the NRC.

7.3.6.2 Load Combinations

The Japanese seismic design criteria do not require that LOCA transient loads be combined with the S_1 or S_2 earthquake loads. The loading from LOCA that occurs after the initial transient loading is combined with the S_1 earthquake load by absolute sum. U.S. practice is to combine all contributions from LOCA loading with the SSE by SRSS. Depending on whether the plant is a PWR or BWR, and what item is being evaluated, it is unknown whether the U.S. or Japanese criteria are more conservative. In the United States, load combination program (as described in NUREG/CR-3660 "Probability of Pipe Failure in the Reactor Coolant Loops of Westinghouse Pressurized Water Reactor (PWR) Plants," [NRC 1985a] and NUREG/CR-4792, "Probability of Failure in Boiling Water Reactor (BWR) Reactor Coolant Piping," [NRC 1988]), it was shown that the primary coolant system supports had a large margin relative to the SSE. This large margin was present due to the fact that the primary system supports were designed for LOCA plus SSE. Thus, the probability of a seismic-induced LOCA was very low. Currently, for existing and new plants, leak before break (LBB) analyses, in conjunction with in-service inspection requirements, limit the requirement for the combination of LOCA and SSE loading conditions for primary systems. Designs that do not combine LOCA and SSE could have significantly less margin for seismic-induced LOCA, if LBB criteria are not met. Quantification of the difference in seismic risk between U.S. and Japanese designs would require studies of the margin in the Japanese designs relative to U.S. designs and probabilistic studies to assess the seismic risk significance.

7.3.6.3 Combination of Earthquake Components

As discussed above, the JEAG 4601 criteria specifies the combination of horizontal and vertical earthquake components by absolute sum. The description is unclear, but if one horizontal component at a time is combined with the vertical component, the result could be un-conservative. As an example, a square anchor bolt pattern subjected to two horizontal components of earthquake would have tension from the two components equal to about 1.4 times the tension from one horizontal component. Because the vertical component of earthquake is a static coefficient based on ½ of the horizontal ground motion, the addition of the vertical component by absolute sum does not increase the bolt tension by any significant amount.

7.3.6.4 Vertical Component of Earthquake

The vertical component of earthquake loading utilized in a dynamic analysis is taken as a static coefficient based on ½ of the horizontal ground motion acceleration. For components mounted on flexible floors, the static coefficient based on ground motion would underestimate the vertical demand on the component. Also, if the component or subsystem is in resonance with the structure, the vertical response could be further amplified. Vertical acceleration usually does not have a significant effect on seismic design, but in some cases the design details could be much different if dynamic vertical response is considered.

8 APPROACHES AND REQUIREMENTS FOR ASSESSING BEYOND-DESIGN-BASIS GROUND MOTION EVENTS

8.1 Background

There is worldwide recognition that there is a small probability that earthquakes may occur that produce ground motions at a nuclear power plant (NPP) site that exceed the design-basis earthquake ground motion of the site. There is a regulatory need to evaluate NPPs for ground motions greater than the design-basis motions, in order to provide confidence that there is no "cliff edge effect" (i.e., that ground motions slightly greater than the design-basis motions do not lead to significant failures in the plant) and to demonstrate that the risk for potential seismic sources is acceptably low.

In addition to identifying potential "cliff edge effects" and assessing seismic risk, there may be numerous other reasons to perform beyond design-basis evaluations. Other reasons for performing such evaluations are an increase in the perceived seismic hazard at the site; investigating the potential for inadequate seismic design (generally due to the vintage of the plant); new technical findings (e.g., vulnerability of selected structures, non-structural elements such as masonry walls, systems, or components such as relays); new experience from actual earthquakes; periodic safety reviews (for countries that require them); and long term operation. The U.S. nuclear power industry (U.S. Nuclear Regulatory Commission (NRC) and licensees) has promptly addressed a number of these issues if and when they arose. For the currently operating U.S. NPPs, issues affecting the performance of structures, systems, and components (SSCs) have been addressed in the Individual Plant Examination of External Events (IPEEE) program (Generic Letter (GL) 88-20, Supplement 4 [NRC 1991]), USI A-46 (GL 87-02 [NRC 1987], [SQUG 1992]), IEB 80-11 [NRC 1980], IEB 79-14 [NRC 1979c], IEB 79-07 [NRC 1979b], and IEB 79-02 [NRC 1979a]. Currently, operating reactors in the United States are reevaluating seismic and flooding hazard and risk in response to perceived increases in seismic hazard at U.S. NPP sites [NRC 2010c, 2011, and 2012c].

Generally, beyond design-basis evaluations have been performed using methodologies of the seismic probabilistic risk assessment (PRA) [ASME/ANS 2003, ASME/ANS 2009, EPRI 1994, EPRI 2009, and NRC 2009b], PRA-based seismic margin assessment (SMA) [NRC 1991a, NRC 2012d], Electric Power Research Institute (EPRI) deterministic SMA [EPRI 1991, EPRI 2003], or design-basis re-constitution.

8.2 U.S. Criteria

Evaluation of beyond design-basis ground motions has been conducted in the United States for all operating NPPs in the IPEEE program [NRC 1991a, NRC 1991b]. In the IPEEE program, licensees conducted either a seismic probabilistic risk assessment (SPRA) to determine a point estimate of core damage frequency or a seismic margin assessment (SMA), which could be either deterministic [NRC 1991a] or probabilistic [NRC 1985b]. The target in either case was to demonstrate a high confidence of low probability of failure (HCLPF) value of 0.3g peak ground acceleration (PGA) for most sites and 0.5g PGA for a few sites in higher seismicity areas.

For new NPPs to be licensed in the United States, a seismic margin of 1.67 times the design-basis ground motion is required to be demonstrated [NRC 1993].

For new NPPs, the U.S. practice for design certification and combined operating licenses is to perform a PRA-based SMA to determine a lower bound on plant capacity referred to as a plant-level HCLPF. In this process, the PRA methodology utilizing event trees and fault trees is applied, but the resulting plant conditional probability of failure is not convolved with a hazard to obtain core damage frequency. While a PRA-based SMA can be used during the design and licensing process for new reactors in the United States, a seismic PRA must be completed and accepted by the NRC prior to loading fuel in a new NPP [NRC 2008b and 2010b]. The certified seismic design response spectrum (CSDRS) is the basis for determining fragilities and HCLPFs of the structures, systems, and components (SSCs) that are modeled. The requirement is that the Certified Design be demonstrated to have a plant level HCLPF at least 1.67 times the CSDRS.

For SSCs located in the balance of the NPP that are designed to site-specific ground motions (i.e., the ground motion response spectrum or GMRS), the same requirement is imposed (i.e., that a plant level HCLPF be demonstrated to be at least 1.67 times the GMRS). This requirement may be satisfied by demonstrating that these SSCs individually satisfy the requirement of HCLPFs at least 1.67 times the GMRS. However, the requirement is for the plant HCLPF to be at least 1.67 times the GMRS, not for individual SSCs to be so.

For new designs, there is likely to be conservatism in the response analysis and design process. A seismic PRA or PRA-based SMA quantifies this conservatism and its effect on safety goals. There is some risk that, for new designs that meet only the minimum design requirements, there may not be sufficient conservatism in some of the most important SSC designs and the desired seismic safety goals may not be achieved. The U.S. addresses this by requiring that a specific margin be designed into the safety-related SSCs. As previously discussed, the U.S. goal is to have a plant level HCLPF that is at least 1.67 times the Certified Design basis and the design certification applicants are addressing this by conducting first a PRA-based SMA and finally a seismic PRA. Japan does not specify a target margin and suggests that the probability of exceeding the design-basis ground motion be examined but does not currently provide any guidance on the decisions that are to be made based on the exceedance probability.

8.3 Japan Criteria

The Nuclear Safety Commission (NSC) Regulatory Guide (RG) of September 19, 2006, recognizes the possibility of beyond design-basis earthquake ground motions occurring and denotes the potential consequences as “residual risk.” Consideration of residual risk is now a requirement. The preferred guidance for determining residual risk is that published by the Atomic Energy Society of Japan (AESJ) [AESJ 2007]. This seismic probabilistic safety assessment (SPSA)[1] standard is currently being translated into English by the NRC as part of the NRC-JNES bi-lateral seismic cooperation program. While the AESJ guidance addresses plant risk through a seismic probabilistic safety assessment (PSA) approach, the guidelines within JEAG 4601-1987 [NRC 1994] do not specifically address the possibility of larger than S_2 ground motions occurring. Reevaluations of existing plants are ongoing and it is understood that larger ground motions may need to be considered. It is clear from the recent good seismic performance of Japan’s NPPs when subjected to larger than design-basis earthquake motions that significant margin is introduced into the design process.

For Japanese new designs for NPPs being offered to other countries, seismic margin and seismic PRA approaches are being implemented to demonstrate margin.

[1] SPSA is equivalent to SPRA in the United States.

8.4 Similarities and Differences in U.S. and Japan Criteria

The United States requires a beyond-design-basis earthquake evaluation for operating and new NPPs. Existing plants have been evaluated for the IPEEE program to verify that no “cliff-edge effects” exist and current activities are underway to reevaluate seismic hazard and risk for the U.S. fleet. New plants are required to demonstrate seismic margin above the SSE ground motion of a plant-level HCLPF of 1.67 times the DBE. This demonstration is to occur during the Certified Design phase and verified once a site has been identified and site-specific features are to be designed. In addition, after construction, an in-plant evaluation is required to assure that no hazards have been introduced during the installation and construction phases and a PRA accepted by the NRC is required prior to fuel loading.

Pre-2006, Japan did not consider beyond-design-basis earthquake ground motions in their evaluations. Post-2006, “residual risk” has been identified as a condition to be considered and the guidance from AESJ has been chosen as the preferred approach to assessments [AESJ 2007]. Regardless, it is not completely clear how beyond-design-basis earthquake motions and the residual risk determined by SPSA will be treated within Japan. It is expected that JEAC 4601 or JEAG 4601 will provide guidance in this regard. To date, evaluations of existing NPPs for newly defined design-basis ground motions have been to existing design criteria (i.e., a design or licensing basis re-constitution at some level of detail).

9 SEISMIC INSTRUMENTATION REQUIREMENTS

9.1 General

9.1.1 Overview of Seismic Data Acquisition System

A seismic data acquisition system (SDAS) is a complete seismic monitoring system consisting of sensors, and data acquisition units (DAUs) that acquire, store, and transmit digital data from one or more systems, including communication hardware and software.

For new nuclear power plant (NPP) applications, the current state-of-practice is for sensors to be accelerometers. In general, the considerations important in an SDAS are discussed below.

- **Robustness.** Equipment should operate reliably over long periods of time (i.e., at least 10 years in the environment of the NPP (site and in-structure)). This environment could include ranges of temperature, high humidity, dust, and/or other conditions. This may lead to requirements for protection against these environmental factors, such as thermal insulation, cases or covers, etc. Instrument output should be unaffected by reasonable changes in magnetic fields and atmospheric pressure; and reasonable levels of radio frequency interference.
- **Measurement type.** Acceleration, velocity, displacement, deformation, strain, and Damage Indicating Parameters (e.g., Standard Cumulative Absolute Velocity (CAV), Japan Meteorological Agency (JMA) Intensity, or others) should be considered. Time varying quantities should be recorded as time histories. Peak values of time varying quantities may also be recorded for specific applications, such as an Automatic Seismic Trip System (ASTS), or manual shutdown. Derived quantities, such as Standard CAV, JMA Intensity, or other Damage Indicating Parameters may be useful in determining the expected level of damage in the nuclear installation and may define whether an operating plant may continue operating or should be shut down within the time frame required by regulations after an earthquake. For purposes of this section, the discussion focuses on acceleration time series.
- **Directions of recorded motions.** In general, for nuclear installations, three directions of motion (two horizontal and the vertical) should be recorded. These triaxial sensors, including the free-field instrument, should be aligned in the principle directions of the installation for ease of use in subsequent evaluations of the structures, systems, and components. It is most convenient if these directions coincide with the principal directions of analytical models of the structures, systems, and components (SSCs).
- **Dynamic range.** The dynamic range of the system is the range of amplitudes that can be accurately measured, bounded below by system and site noise or digital resolution, and bounded above by the sensor. The dynamic range is typically defined as the signal to noise ratio. Dynamic range is measured in decibels (dB) and equivalent bits.
- **Frequency range or bandwidth.** The frequency range is the range of frequencies that can be accurately reproduced by the recorded data. The overall bandwidth is a function of the system (i.e., sensors, cabling, and digitizer bandwidth). Minimum frequency range is 0.02 to 50 Hz with DC to 100 Hz desirable. Typically, the low frequency range is at 0.01 Hz. The minimum sampling rate should be 200 samples per second.

- **Cross-axis sensitivity.** The cross-axis sensitivity is the sensitivity of the measurements in one direction to motions in the other two directions. Cross-axis sensitivity should be as low as possible. It is usually measured as a ratio of amplitude of motion to that of the main direction of interest.
- **Absolute timing accuracy.** The recorded motion from multiple instruments should be based on a common time scale (instruments should be interconnected). These records are appropriately correlated in time for further data assessments. For example, in the free-field, the assessment of ground motion incoherency could be made based on the recorded data from an array of instruments. On the foundation, rotations of the foundation (rocking and torsion) can be derived from multiple instrument recordings to permit post-earthquake dynamic analyses of structures subjected to appropriately correlated base translations and rotations. In-structure instruments recording motions correlated in time with free-field and basemat motions can be interrogated to determine structure dynamic characteristics from transfer functions derived from the Fourier transforms of the recorded motions.
- **Pre-event memory.** Pre-event memory times should be sufficient to capture the P-wave motions, when the sensor is triggered to save data by the S-wave motions. A minimum of 30 seconds is recommended.
- **Recording capacity.** Recording capacity should be adequate to capture the entire free-field record and the free vibration response of the structure after the strong shaking has reached a minimum level.
- **Multiple event recordings.** There should be adequate provisions to permit recording and data capture of multiple events that may occur within a short time interval, such as a few hours.

Table 9-1 summarizes the performance characteristics of seismic instrumentation systems from the 1970s to the current state-of-practice. Although Table 9-1 and the ensuing discussion emphasize United States (U.S.) practice, these are general requirements, which are or will be adhered to in many countries.

- Early vintage systems were analog with very limited capability to meet the current objectives as itemized above. Early vintage systems may have included response spectrum recorders – basically scratch plate devices to directly record response spectral ordinates for comparison with design-basis earthquake ground motion parameters. It should be noted that scratch plates in NPPs were intended for post-earthquake engineering reviews and not for assessment of shut down or restart requirements. Because of the two-dimensional nature of scratch plates, the error in measurement of ground motion is nearly universally unconservative because out of plane motion causes the arms to lift off the plate (and not record) or to dig into the plate (and not move freely).
- The evolution from the early vintage systems to today is shown in Table 9-1. In the United States, guidance for seismic instrumentation of NPPs was first issued in the early 1970s and denoted a Safety Guide. In April 1974, a revision of the Safety Guide was issued as NRC Regulatory Guide (RG) 1.12, “Nuclear Power Plant Instrumentation for Earthquakes,” Revision 1. NRC RG 1.12, Revision 1, was revised and issued as Revision 2 in March 1997. Currently, NRC RG 1.12 is under revision. In all cases, revisions were made to accommodate changes in the state-of-knowledge of earthquakes and their characteristics and changes in the areas of instrumentation systems.

- The current state-of-practice for SDAS in NPPs is defined in Table 9-1, Column 5, titled “2009 Commercial NPP Systems.” Considering the characteristics itemized above, systems identified by the parameters of Column 5 meet or exceed all of the requirements specified.[1]
- Table 9-1, Columns 6 and 7 identify minimum requirements for two classes of seismic instrumentation (Class A and B) as defined by the U.S. Advanced National Seismic System (ANSS), which is run by the U.S. Geological Survey. Class A instruments are sensors and DAU at or near the state-of-the-art. Class B is one step down from the state-of-the-art. Both Class A and B closely match the units described in Column 5, with some exceptions. The system requirements of Columns 5, 6, and 7 indicate the current state-of-practice.

Table 9-1 NPP seismic instrumentation performance (Courtesy of Prof. Nigbor, 2010)

Performance Parameter	70s-Vintage, Kinemetrics SMA-3	RG 1.12 Rev. 2 1997	ANSI/ANS 2.2-2002	2009 Commercial NPP Systems	Advanced National Seismic System Class B	Advanced National Seismic System Class A
Dynamic Range (dB)	40	60	60	86-108	>86	>110
Dynamic Range (equiv. bits)	7	10	10	16-18	≥16	≥20
Full-Scale (g)	1	1	1	3-4	3.5	3.5
Frequency Range (Hz)	0.2-30	0.2-50	0.02-50	0-50	0.1-35	0.02-50
Sample Rate (samples/ sec)	Analog	200	200	200+	200(min.)	200(min.)
Cross-Axis Sensitivity (g/g)	0.03	Not specified	0.03	0.01	0.01	0.01
Absolute Timing Accuracy (msec)	No timing	Not specified	Adequate to differentiate main shock	<1 with GPS	<1	<1
Pre-event Memory (seconds)	0	3	3	99 or greater	≥60	≥60
Recording Capacity (minutes)	10	25	25	>100	≥60	≥60

[1] Currently Kinemetrics, Syscom, and GeoSig are certified for NPPs in the United States.

9.1.2 Automatic Seismic Trip Systems

There are many factors to consider when deciding whether an automatic seismic trip system (ASTS) is needed or appropriate for a NPP. Several considerations and their advantages and disadvantages are:

- **The level, frequency, and duration of earthquake activity at the NPP site.** An automatic system is rarely justifiable for sites in areas of low seismic activity. Moderate to high seismic areas are more likely to have an ASTS.
- **The seismic capacity of NPP systems compared to the probability of the design-basis ground motion occurring at the site.** Automatic systems could be used as an additional protective measure, particularly in the case of increases in the perception of the seismic hazard at the site or when the seismic design-basis has been increased.
- **Safety considerations relating to spurious scrams.** An automatic system should not be used in places with high levels of ambient noise, including noise induced by other plant equipment; spurious scrams may have a negative impact on the perception of the public on the reliability of the plant, especially if it leads to a loss of electricity in the public's daily life.
- **Effects of the superposition of earthquake acceleration on the seismic transient induced by an automatic scram.** In some cases, a combination of earthquake acceleration and seismic transient may be more challenging to plant safety than the scenario of an earthquake affecting the plant at full power.
- **Broad ranging safety issues relating to the region if the plant shuts down immediately following an earthquake.** In some regions with a limited electricity grid and few seismically qualified power generation plants, the availability of power in an emergency could be essential, and an automatic scram should therefore be used only if it is ascertained that there is a challenge to the safety of the plant. This does not, however, imply that the Operating Basis Earthquake (OBE) ground motion is exceeded in which case the regulatory requirements apply.
- **Level of operator confidence and reliability.** For a nonautomatic system, the operator plays an important part in the decisions on post-earthquake actions and therefore should be adequately trained for this contingency.
- **Operator acceptance and appreciation.** For large ground motions, in times of high stress with many things happening onsite and offsite (such as concerns for family and friends), operators may appreciate the decision being automated.
- **Public acceptance.** Public acceptance is an important aspect which may influence the decision on the approach to adopt. The installation of an automatic trip system may be perceived either positively as an additional safety system or negatively as a lack of confidence in the seismic design level and the seismic safety of the installation. Public opinion depends heavily on the level of experience and education of the population with regard to earthquake events. The impact of spurious trips—if perceived directly by the public due to a perturbation in the supply of electricity—will probably impact negatively on the public perception of the reliability of the plant.

In the United States and Japan, there is some similarity in requirements for ASTS insofar as that for NPPs in areas of high seismicity, ASTS has been employed. In all NPPs in Japan, a country of high seismic areas, ASTS is required. Whereas, in the United States, only two sites, both in areas of high seismicity, have ASTS. For NPPs sited in moderate seismic areas, ASTS is not required.

9.2 U.S. Practice

9.2.1 Seismic Instrumentation in the United States

Several events mark changes in the seismic instrumentation requirements for existing NPPs in the United States.

9.2.1.1 Requirements and guidance from 1973 to 1997

10 CFR 100 Appendix A

On November 13, 1973, the Atomic Energy Commission (AEC), predecessor to the NRC, issued Appendix A, "Seismic and Geologic Siting Criteria for Nuclear Power Plants," to Title 10, Part 100 of the *Code of Federal Regulations* (10 CFR Part 100), "Reactor Site Criteria," effective December 13, 1973. The regulations require licensees to provide suitable instrumentation so that the seismic response of NPP features important to safety can be determined promptly to permit comparison of such response with that used as the design basis (Paragraph VI(a)(3)), and to shut down the NPP if vibratory ground motion exceeding that of the OBE ground motion occurs (Paragraph V(a)(2)).

The Supplementary Information to the final regulation (38 FR 31279, Item 6e) included a statement that a footnote was added to Section 50.36(c)(2) of 10 CFR Part 50, "Domestic Licensing of Production and Utilization Facilities," to assure that each NPP is aware of the limiting condition of operation which is imposed under Section V(2) of Appendix A, "Seismic and Geologic Siting Criteria for Nuclear Power Plants," to 10 CFR Part 100 (if vibratory ground motion exceeding the OBE ground motion occurs, shutdown of the NPP will be required). At that time, it was the intention of the Commission to treat the OBE ground motion level as a limiting condition of operation. From the statement in the Supplementary Information, the Commission directed applicants to specifically review 10 CFR Part 100 to be aware of this intention in complying with the requirements of 10 CFR 50.36, "Technical Specifications." Thus, the requirement to shut down if ground motion exceeding the OBE level occurs was expected to be implemented by being included among the technical specifications submitted by applicants after the adoption of Appendix A. In fact, many applicants did not include OBE ground motion exceedance shutdown requirements in their technical specifications.

Regulatory Guide 1.12

Revision 1 of RG 1.12, "Instrumentation for Earthquakes," was issued in April 1974[2] and describes seismic instrumentation acceptable to the AEC staff as satisfying the requirements of Appendix A to 10 CFR Part 100. The guide endorses, with exception, a variety of earthquake instrumentation types mounted on both structures and equipment that are specified in American Nuclear Society Standard ANSI N 18.5, "Earthquake Instrumentation Criteria for Nuclear Power Plants."

The obligation of the licensee was as stated. If the plant did not shut down as a direct or indirect result of the earthquake shaking and if the licensee did not include OBE ground motion shutdown requirements in their technical specifications, then the decision to shut down the plant was to be made by the NRC when provided all available information. There is no other provision in 10 CFR

[2] The current version of this RG is [NRC 1997]

Part 50 that requires the licensee to shut down the plant. Therefore, it has been determined that the shutdown is an NRC initiated event.

9.2.1.2 Requirements and guidance from 1997 to Present

10 CFR 50, Appendix S

The NRC issued Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants," to 10 CFR Part 50, "Domestic Licensing of Production and Utilization Facilities," on December 11, 1996 (61 FR 65157), effective January 10, 1997. Appendix S to Part 50, "Earthquake Engineering Criteria for Nuclear Power Plants," requires licensees to provide suitable instrumentation so that the seismic response of NPP features important to safety can be determined promptly to permit comparison of such response with that used as the design-basis (Paragraph IV(a)(4)):

> *"(4) Required Seismic Instrumentation. Suitable instrumentation must be provided so that the seismic response of nuclear power plant features important to safety can be evaluated promptly after an earthquake."*

and to shut down the nuclear power plant if vibratory ground motion exceeding that of the OBE ground motion or significant plant damage occurs (Paragraph IV(a)(3)):

> *"(3) Required Plant Shutdown. If vibratory ground motion exceeding that of the Operating Basis Earthquake Ground Motion or if significant plant damage occurs, the licensee must shut down the nuclear power plant. If systems, structures, or components necessary for the safe shutdown of the nuclear power plant are not available after the occurrence of the Operating Basis Earthquake Ground Motion, the licensee must consult with the Commission and must propose a plan for the timely, safe shutdown of the nuclear power plant. Prior to resuming operations, the licensee must demonstrate to the Commission that no functional damage has occurred to those features necessary for continued operation without undue risk to the health and safety of the public and the licensing basis is maintained."*

Licensees of NPPs that are subject to the earthquake engineering criteria in Appendix S to 10 CFR Part 50 are required by 10 CFR 50.54(ff), "Conditions of Licenses," to shut down the plant if the criteria in Paragraph IV(a)(3) of Appendix S are exceeded (OBE vibratory ground motion or significant plant damage). Shutdown initiated by significant plant damage was not included in Appendix A to Part 100.

Regulatory Guide 1.12 Revision 2

Revision 2 of RG 1.12, "Nuclear Power Plant Instrumentation for Earthquakes," was issued in March 1997. This RG describes seismic instrumentation acceptable to the NRC staff to satisfy the requirements of Appendix S to 10 CFR Part 50. The consensus standard available at that time, ANSI/ANS.2.2-1988, "Earthquake Instrumentation Criteria for Nuclear Power Plants," was not endorsed by the NRC staff because it utilized a variety of earthquake instrumentation types mounted on both structures and equipment[3]. Experience has shown that data obtained from instrumentation located on equipment and piping is contaminated by the vibratory motion

[3] The most recent version of the standard is [ANSI/ANS 2002].

associated with normal plant operation. Therefore, the NRC issued a government-unique standard (RG 1.12) describing a single instrument type (triaxial time-history accelerograph), which is mounted on structures, not equipment

Regulatory Guide 1.166

RG 1.166, "Pre-Earthquake Planning and Immediate Nuclear Power Plant Operator Post-Earthquake Actions," issued March 1997, provides guidance acceptable to the NRC staff for a timely evaluation of the recorded instrumentation data after an earthquake, and for determining whether plant shutdown is required.

- The evaluation to determine whether the OBE ground motion was exceeded should be performed using data obtained from the three components of the free-field ground motion (i.e., two horizontal and one vertical). The evaluation may be performed on uncorrected earthquake records. It was found in a study of uncorrected versus corrected earthquake records (see EPRI NP-5930 [EPRI 1988]) that the use of uncorrected records is conservative. The evaluation should consist of a check of the response spectrum and CAV and a check on the operability of the instrumentation. This evaluation should take place within 4 hours of the earthquake.

- Pre-earthquake planning includes the selection of a cross-section of safety and non-safety related equipment to be inspected in the event the earthquake exceeds the OBE ground motion using the guidelines in Section 5.3.1 of EPRI NP-6695, "Guidelines for Nuclear Plant Response to an Earthquake" [EPRI 1989]. The results of baseline visual inspections of this equipment should be documented in reports, photographs, etc.

- The evaluation to determine whether "significant plant damage" has occurred should be performed using the guidelines that are specified in Sections 4.3.1 (with noted exception) and 4.3.2 of EPRI NP-6695. The inspections should be similar to those performed by plant operators during their normal daily rounds. It is important that the walkdown inspections be performed by plant operators familiar with the equipment to be inspected in order to know if the physical appearance, leak rates, vibrations levels, and other related attributes have changed.

Regulatory Guide 1.167

RG 1.167, "Restart of a Nuclear Power Plant Shutdown by a Seismic Event," [NRC 1997] provides guidance acceptable to the NRC staff for performing inspections and tests of NPP equipment and structures prior to restart of a plant that has been shut down by a seismic event.

IAEA Safety Report 66, "Earthquake Preparedness and Response for Nuclear Power Plants," was recently published in 2011, [IAEA 2011]. This report provides an overall methodology addressing pre-earthquake planning and post-earthquake actions for NPPs and includes lessons learned from activities related to the restart of the Kashiwazaki-Kariwa NPP. This methodology builds on the U.S. NRC RGs 1.166 and 1.167 and EPRI NP-6695, including recommended actions to be performed if ground motion exceeds the equivalent of the SSE ground motion. This is the first time that beyond SSE ground motions are specifically addressed in nuclear guidance. An update to EPRI NP-6695 [EPRI 2012a] was completed in 2012, which took into account some aspects of IAEA Safety Report 66.

Standard Review Plan Section 3.7.4, "Seismic Instrumentation"

NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants," (also known as the SRP) was updated and published as Revision 2 in 2007. Section 3.7.4 of the SRP incorporates Revision 2 of RG 1.12 and RG 1.166.

The requirements and guidance of 10 CFR 50, Appendix S, SRP Section 3.7.4, and related RGs 1.12, 1.166, and 1.167 define the current state-of-practice in the United States.

If an NPP license is governed by 10 CFR 50, Appendix S, the above criteria apply. In this case, the obligation of the licensee is to determine if the OBE ground motion exceedance has occurred and, if so, take appropriate action to shut down the plant, if the plant did not shut down as a direct or indirect result of the earthquake shaking.

9.2.1.3 Current Criteria for New Plants

New plants, as characterized by their Design Certification Documents, adhere to the requirements and guidance discussed above, but with a number of exceptions or issues not addressed.

The NRC is considering a revision to RG 1.12 to incorporate changes in the state of the practice in seismic instrumentation and the lessons learned from actual earthquake experiences, particularly in Japan. A technical approach document, "Technical Approach, Revision of Regulatory Guidance for Nuclear Power Plant Instrumentation for Earthquakes," [ISL 2012] is under review by the U.S. NRC as a first step in the revision process. In parallel, the American Nuclear Society (ANS) has formed a committee to revise standard ANSI/ANS 2.2, "Earthquake Instrumentation Criteria for Nuclear Power Plants."

9.2.2 Automatic Seismic Trip Systems in the United States

In the United States, the implementation of an ASTS is not addressed in the regulations. Hence, its implementation would be a plant-specific consideration and decision.

In the early 1980s, the NRC funded a study performed by Lawrence Livermore National Laboratory, "On the Advisability of an Automatic Seismic Scram" [NRC 1981]. The approach was to use available models and data to assess the change in a risk metric (specifically core damage frequency (CDF)) when an ASTS is installed at a plant. Existing information on plant accident sequences and plant behavior was extracted from Safety Analysis Reports and used in conjunction with probabilistic risk assessment (PRA) systems models to assess the advantages of an ASTS. Specific information on timing of loading conditions induced in systems due to reactor trips was used along with the timing of earthquake-induced stresses and deformations to assess the impact. Also, the required time of control rod insertion played a role. A realistic analysis for a specific site and plant requires site-specific and plant-specific data. Such data was not available for the study and generic data for a hypothetical plant was used.

A decision-tree modeling approach was used to compare the risks (CDF) of employing an ASTS compared to not employing an ASTS. For the hypothetical plant, and using data from many sources, the results showed that an ASTS would reduce the probability of an earthquake-induced core damage event by about a factor of three. Partially offsetting this advantage was the disadvantage of inadvertent reactor trips. This study is 30-years old. All aspects of NPP design have significantly changed in the ensuing three decades. If it were deemed necessary to revisit the decision to require ASTS for all plants or a subset of plants, a study using similar tools for current designs and existing sites could be performed to provide additional information for decisionmaking.

Only two plants in the United States have ASTS: Diablo Canyon Power Plant and San Onofre Nuclear Generating Station. Both are in areas of high seismicity.

9.3 Japan Practice

9.3.1 Seismic Instrumentation in Japan

There are no legal seismic instrumentation requirements in Japan separate from those for the ASTS (Section 9.3.2). In addition to the seismic instrumentation requirements for ASTS, the licensee decides on the number and type of instruments and their placement in the free-field and on the structures (with approval of the regulator). Generally, the focus is on instruments to record acceleration time histories. Also, there is a focus on recording three components of motion. Typical practice in Japan is the installation of free-field instruments on the soil or rock free surface and downhole instruments to a depth of 300 or 400 m. Downhole instruments are intended to provide information on motions at depth to compare with design-basis ground motion specifications that are at depth on hypothetical outcrops. On-foundation and in-structure instruments range in number from 5 to many 10s of instruments per unit.

Generally, the instruments are maintained regularly and tested for operability multiple times per year. If instrumentation systems do not meet the guidelines of Section 10.1, upgrades will be implemented over the next years. These upgrades will especially address lessons learned from the NCO earthquake with regard to not over-writing main shock records with aftershock data and ensuring data transmission capability to decisionmakers.

9.3.2 Automatic Seismic Trip Systems in Japan

NPP units in Japan are required to have in place an ASTS. Appendix A contains the relevant legal requirements and the implementation guidelines as stated in JEAG 4601-1987.

Overall requirements are for the system to have redundancy, independence from other reactor systems, qualification to seismic plus other loading conditions, testability (to assure operability), and logic for shutdown that accounts for multiple exceedances before shutdown is initiated. The trigger point for shutdown is peak accelerations less than or equal to about 70 to 90 percent of the S_1 in terms of pre-2006 nomenclature and likely about 70 to 90 percent of the S_d (the OBE ground motion equivalent).

The decision to require ASTS for NPP units in Japan is supported by a number of factors:

- ***Lessons learned from actual earthquakes.*** The example of a lesson learned from an actual earthquake is the experience of Mr. Yoshitaka Irisawa, Shift Supervisor, Kashiwazaki-Kariwa Nuclear Power Plant, Unit 4, when the Niigata-ken Chūetsu-Oki earthquake occurred on July 1, 2007 [Irisawa 2009]. In summary, Mr. Irisawa praised the “Success of the Scram” as it promptly shutdown the reactor and consequently calmed the operating staff, which led to their being able to perform their other duties in a calm and professional manner. Mr. Irisawa and staff were commended by the Japan Society of Mechanical Engineering for their outstanding contribution to the reactor safety.
- ***Scramability.*** The term “scramability” refers to the demonstrated ability of the control rods to be inserted during the earthquake shaking of the plant. This phenomenon encompasses

two aspects of plant shutdown. The first is the ability of the control rods to be inserted into the core in the required time taking into account the seismic demand imposed on the core, control rods, and control rod insertion system. This seismic demand includes relative deformations of the core on the control rod channels. The second aspect is the load combination of the reactor scram and loads imposed by transients or other events. Both aspects are important. An automatic seismic system set at trigger levels less than the S_d (OBE) or S_s (SSE) is expected to cause control rod insertion to be completed before strong shaking occurs and before loading conditions from consequential events are imposed on the core, control rods, and control rod insertion system.

Given the performance history and the judgment of Japanese regulators and licensees, it is highly unlikely that the requirement for ASTS will be relaxed in the future.

9.4 Similarities and Differences in U.S. and Japan Practice

Seismic Instrumentation

The U.S. practice for seismic instrumentation is generally significantly less rigorous than that of Japan[4]. As described in Section 9.2, seismic instrumentation for existing NPP units appears to be less prescriptive with more decisions left to the licensee. This practice meets the legal requirements of the United States. For new plants, a review of the Certified Design documentation generally commits to RG 1.12, Revision 2, requirements, but with some exceptions. However, the NRC is considering a revision to U.S. NRC RG 1.12, which is likely to address issues such as those itemized in Section 9.1 that are not addressed in the current RG. In addition, typical new plant configurations have many structures founded on a very large basemat (termed a nuclear island), which adds new elements to the decisionmaking process (e.g., the number and placement of sensors may change).

The practice in Japan is to install numerous instruments to capture the free-field and in-structure motions due to actual earthquakes. They also install downhole instruments. Their practice adheres to many of the principles of Section 9.1 with commitments to improve those areas where improvement is necessary. Due to the high seismic activity in Japan, these practices are prudent.

A more robust approach to seismic instrumentation systems in the United States may be prudent particularly given the experience at the North Anna Power Plant (NAPP) after the 2011 Mineral, Virginia, earthquake [NRC 2013]. If the ground motion and induced structure response during an earthquake are not accurately recorded, the structural response cannot be determined with confidence, or if the ground motions cannot be determined quickly (as was the case at NAPP), very difficult shutdown and restart decisions will need to be made by the NRC and the licensee. At least a minimum level of seismic instrumentation with assurance of its operability when an earthquake occurs should be ensured.

Automatic Seismic Trip System

The philosophy of requiring automatic scram systems differs for the United States and Japan. In the United States, on a plant-specific basis (typically, in high seismic areas), automatic scram systems may be required for some NPPs. In Japan, ASTS is required for all NPP units. It should be noted that

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[4] The only notable exception is the U.S. requirement of a nuclear-specific certification of seismic monitoring instrumentation that accounts for the unique needs of NPPs, such as radiation resistance.

all Japanese plants are located in areas of high seismicity. It is unlikely that the respective philosophies will change in the near future.

10 SUMMARY AND CONCLUSIONS

Globally, the two countries considered to have the most developed seismic design standards and calculational methods for nuclear facilities with emphasis on nuclear power plants (NPPs) are the United States and Japan. Thousands of person-years of effort over 5 decades have been devoted to developing these standards and methods for nuclear facilities. In addition, Japan has experienced several earthquakes that have directly affected NPPs with ground motions exceeding the S_1 or S_2, and in some cases significantly exceeding the S_2. In these cases, minimal or no damage to safety-related structures, systems, and components (SSCs) was observed. Even for the 2011 Tōhoku earthquake, evidence suggests that damage of safety-related SSCs due to ground motion was not significant at any of the affected NPPs. Further, field experiments and laboratory testing in the United States, Japan, and Taiwan over this same period have illuminated aspects of the standards and the conservatism contained therein. All of these factors have led to the evolution of seismic design standards and calculational methods over the five decades. The lesson learned from the aggregate of these experiences is that seismic design of NPPs in the United States and Japan has been demonstrated to be highly effective when tested by actual earthquake shaking.

In this context, a review and comparison of the state of practice in the United States and Japan is appropriate. The United States, Japan, and other countries can learn from these experiences and introduce appropriate changes to their seismic design standards and calculational methods. It is important to note, however, that seismic hazard assessment and seismic design must always be considered in the context of the seismo-tectonic environment in which the country exists. Nearly the entire country of Japan is situated in an area of high seismicity and both subduction and active crustal mechanisms are at work. The United States, in contrast, is highly varied with seismicity rates that range from very high to very low across its territory and with nearly every seismo-tectonic environment found within its borders. It should be expected, therefore, that differences in assessment, design, and regulatory approaches exist between the two countries.

Seismic design and qualification is a complicated process with a significant number of interrelated parameters and required analytical steps. The comparison of different seismic design and qualification criteria requires the careful consideration of each of these interrelated parameters and analyses. As might be expected, the overall conclusion from this review of the U.S. and Japanese criteria is that there are a number of more conservative and less conservative parameters within each of these national seismic criteria with respect to the other. The identification of these differences provides valuable insights into both the reasons for differing responses to actual earthquakes within the United States and in Japan, and the areas where changes may be recommended in future revisions to the seismic design codes and standards within both countries. Because of the analytical and computational complexities inherent in the seismic design process, it is not possible to conclude which criteria (U.S. or Japan) may be more conservative overall without examining specific designs for a unique plant application.

There is also the question of the conservatism integrated into the process of defining the design-basis ground motions starting with the seismic hazard analysis. The very different approaches for seismic hazard analysis, as well as the fault displacement hazard analysis, in the United States and Japan would need to be analyzed to fully appreciate the conservatism that may exist in a design of a plant located at a particular site. Currently both countries are reevaluating seismic hazard at operating reactor sites. It is worth noting, though, that the use of the term “conservative” has less meaning for the probabilistic seismic hazard assessment (PSHA) used in the United States. First, the objective of PSHA is an accurate assessment of the probability of a particular ground motion, with a full accounting of uncertainty. Second, because all ground motions have associated probabilities, it is unclear what an assessment of conservatism would be against in terms of the baseline value.

Conservatism in NPP design is appropriate. However, excess conservatism in one aspect of the design may have a negative impact on the overall safety of the plant. An example of this point is the conflict between adding piping supports to increase the dynamic characteristics (natural frequencies) of a piping system, which simultaneously increases the stress in the piping system caused by thermal loads. The overall effect on safety is not apparent.

One way to quantify differences in implementation of the seismic analysis, design, and qualification processes for the United States and Japan is to perform a pilot study whereby a given design is compared step-by-step to quantify the conservatism or unconservatism in each step. An abbreviated effort in this regard is being performed for the International Atomic Energy Agency Extra-Budgetary Kashiwazaki-Kariwa Research Initiative for Seismic Margin Assessment (or KARISMA) project.

The key comparisons of Sections 3 through 7 are summarized in Tables 10-1 through 10-5, which are expansions of those presented by Kassawara [2008]. Sections 8 and 9 are summarized below.

In summary, some elements of the seismic design process can be clearly defined as being more or less conservative and the relative conservatism of other elements cannot be defined without more specific comparisons. The comparison of Japan Pre-2006 and the U.S. Pre-2007 criteria is significant because it is these criteria that applied to the analysis, design, and qualification of operating plants in Japan (prior to any upgrading that occurred because of the new seismic design-basis earthquake ground motion definitions) and the United States, and are summarized below. In both cases, experience has shown that the level of conservatism in seismic design in NPPs in the United States and Japan against any particular design ground motion is high.

Elements of the seismic analysis, design, and qualification processes for which Japan is more conservative than the United States (Japan Pre-2006 compared to U.S. Pre-2007)

- Structure damping values used in linear analysis are lower in Japan than in the United States.
- Damping values for some equipment, components, and piping are lower in Japan than in the United States.
- Implementation, testing and maintenance of modern seismic instrumentation systems are required in Japan and not in the United States.
- Testing for equipment seismic performance and fragility is performed in Japan, while proof testing is performed in the United States.

Elements of the seismic analysis, design, and qualification processes for which the United States is more conservative than Japan (Japan Pre-2006 compared to U.S. Pre-2007)

- All safety-related SSCs (Seismic Category I) are designed to safe shutdown earthquake (SSE) ground motion (by comparison, under the Japan criteria safety-related equipment is designed to S_1 and only a subset is assessed for functionality under the S_2 ground motion).
- Soil-structure-interaction (SSI) analyses to determine the structure response are performed for soil and soft rock sites. The structure response used for design, including as

input to subsystems, is defined as the envelope of the responses for three soil profile cases in the United States; only a best estimate soil profile is considered in Japan.

- In-structure response spectra are developed with peaks broadened ±15 percent in the United States as compared to ±10 percent in Japan.
- Three components of earthquake ground motion are considered simultaneously in the SSI analyses.
- All combinations of loss-of-coolant accident (LOCA) loadings are combined with the SSE ground motions.
- Equipment qualification testing is required.
- Beyond design-basis ground motion evaluations are required for new plants; acceptance criteria for new plants, plant-level high confidence of low probability of failure (HCLPF) values must be greater than 1.67 times the design-basis ground motion. A seismic probabilistic risk assessment (SPRA) is required prior to loading of fuel in new reactors.
- For existing plants, similar (though early vintage) beyond design-basis assessment procedures were previously implemented and assessments performed during the Individual Plant Examination of External Events (IPEEE) program in the last 1990s. These assessment approaches subsequently matured and expanded into a number of tools that now exist for a variety of design, assessment, and operational uses. The current risk-informed performance-based operational and regulatory framework is a direct result of that early work and the lessons learned. Licensees are now in the process of reassessing the seismic hazard for all U.S. operating reactors and the U.S. Nuclear Regulatory Commission (NRC) will use the beyond design-basis tools for those NPPs whose new estimated ground motion exceeds the original design.

Elements of the seismic analysis, design, and qualification processes for which the relative conservatism is currently unknown (Japan Pre-2006 compared to U.S. Pre-2007)

- Probability of occurrence of peak values of design ground motion (e.g., Peak Ground Acceleration (PGA), Peak Ground Velocity, and Peak Ground Displacement) for the operating reactors is currently unknown, although analyses to determine this information are underway.
- In Japan, maximum of static and dynamic loads are used for design (e.g., a static loading of 0.6g for structures and 0.72g for equipment, piping, etc.)
- In the United States, the minimum design ground motion at foundation level in the free-field has a minimum PGA of 0.1g anchoring a spectral shape appropriate for foundation level (outcrop or in-column motion). Most commonly a RG 1.60 ("Design Response Spectra for Seismic Design of Nuclear Power Plants") spectrum anchored at a 0.1g PGA is used.
- Automatic Seismic Trip Systems are required for all NPP units in Japan.

Elements of the seismic analysis, design, and qualification processes that are very favorable for Japan in reducing uncertainty in dynamic behavior of SSCs and verifying seismic capacity

- Extensive testing program to verify behavior of soil-structure systems (SSI phenomena and methods of analysis)

- Extensive testing program to define the stiffness, nonlinear behavior, and capacity of structure elements, e.g., shear walls
- Extensive testing program to define the behavior of equipment, piping, and other components

Other elements of the seismic criteria are judged to be similar in conservatism.

Perspective

Historically, both the U.S. and the Japanese practices have used deterministic approaches in all aspects of the seismic analysis, design, and regulation. However, over the years, and particularly in connection with the new reactors, the U.S. practices are moving toward a more performance-based, risk-informed regulatory framework. The Japanese practice has recently begun to look at very limited aspects of risk-informed considerations. Its practice is still basically deterministic. The following describes how the risk-informed aspects are currently being used and provides a brief comparison of the two practices.

Japan has introduced the “residual risk” concept in 2006; however, the approach taken in seismic hazard assessment and seismic design is still inherently deterministic in nature. As in the most deterministic practices, the focus in Japan is on assuring that a high level of conservatism exists at every step in the design process, such that Japanese NPPs have significant margin above the design-basis earthquake (DBE) ground motion used. There is an assumption that the DBE ground motion used is sufficiently rare for the site of interest.

By contrast, the U.S. uses a mixed approach. For existing operating NPPs, meeting the NRC’s seismic-safety regulations still means meeting a complex set of deterministic regulations that are demonstrated by deterministic evaluations. This includes how the design-basis earthquake (the SSE) still in use was selected, although a probabilistic reevaluation of that SSE is now under way for all existing plants. For new designs, the same set of deterministic regulations, demonstrated by deterministic analyses, is still in place, except that the selection of the SSE for a new plant must follow a probabilistic seismic hazard approach tied to a specific annual frequency of exceedance. What is new is that the regulatory evaluation of the design, which uses deterministic criteria similar to those used for the existing operating plants, is supplemented by a risk-informed and performance-based evaluation of the seismic adequacy of the plant-as-a-whole. This evaluation provides a clearer way to understand conservatisms inherent in the design and provides an opportunity to risk-inform the entire design practice.

These two philosophies are so different that the relative conservatism of the outcomes of the two approaches cannot be known *a priori*. The conservatism of any regulatory framework for an NPP can only be assessed through a comparison of the true response of the NPP against the true hazard at its site. A seismic probabilistic risk assessment provides a means to evaluate the conservatisms.

Although for new plants the United States relies in part on a performance-based, risk-informed framework, the process of seismic analysis, seismic design, and seismic qualification of SSCs is deterministic by choice and the practicality of design. Deterministic procedures (methods and parameter values) are developed and evaluated to assure that the implementation of seismic analysis, seismic design, and seismic qualification for SSCs leads to SSC seismic performance that meets the risk guidelines.

A comparison of the results of the deterministic seismic analysis, design, and qualification process step-by-step is less satisfying than a comparison of SPRA results; however, it is still a valuable exercise. The end result is a comparison of the design loading conditions for SSCs, including loads, in-structure response spectra (ISRS) for qualification of equipment, components, and distribution systems, and other design quantities. This comparison could be conditional on the DBE or include the effects of the DBE. The end result quantifies the degree of relative conservatism introduced in various steps of the seismic analysis chain by U.S. procedures compared to the procedures of Japan. The end result could also be interpreted in the risk framework as a surrogate for core damage frequency (CDF) or large early release frequency (LERF), such as onset of inelastic deformation. This is a very valuable and practical assessment process recognizing the multi-disciplinary nature of the process.

For the above reasons, the discussions in this document are framed to provide clarity and insights into the similarities and differences of the two regulatory approaches and frameworks. This document does not, and cannot, provide a strict "apples to apples" comparison of each step in the process.

Table 10-1 Elements of design-basis earthquake ground motion in the United States and Japan in operating and new reactors

Elements of the Process	Japan Pre-2006 Design	Japan Post-2006 Design	U.S. Pre-2007 Design	U.S. Post-2007 Design
Seismic Hazard Analysis	Deterministic, Empirical GMPE (ground motion prediction equation), and Fault Simulation	Deterministic, Empirical GMPE, and Fault Simulation; Probabilistic for Residual Risk Evaluation	Deterministic with Supporting Probabilistic Analysis	PSHA with Application of senior seismic hazard analysis committee (SSHAC) Level 3 or 4; SSHAC level 2 studies may be used to update regional SSHAC 3 or 4 studies if the underlying regional model is still valid.
Spectral Shape	Standard Response Spectra Shapes (based on M, d)	To be determined	Generally RG 1.60 or similar spectrum	Probabilistic seismic hazard analysis (PSHA) with site response analysis incorporated as appropriate
Design-Basis Ground Motion	S_1 Maximum Design Earthquake and S_2 Extreme Design Earthquake	S_s (Design-Basis Ground Motion) and S_d (Elastic Design Ground Motion)	Operating Basis Earthquake (OBE) Ground Motion and Safe Shutdown Earthquake (SSE) Ground Motion	Certified Seismic Design Response Spectra (for Certified Design) and ground motion response spectra (GMRS) site-specific hazard with minimum (below) used for non-standard design and of assessing suitability of certified design (and to design the balance of plant.
Control Point	Free Surface of Base Stratum (Rock)	Free Surface of Base Stratum (Rock)	Top of Grade (TOG) or Rock Outcrop for Layered Sites	Actual or hypothetical rock outcrop or TOG as defined in RG 1.208
Spatial Components	Horizontal + Vertical (statically)	Horizontal + Vertical	Three Components	Three Components
Minimum Earthquake Scenario	M = 6.5, D_{hyp} = 10			
Minimum Ground Motion	Static Applied Load	Static Applied Load		10 CFR 50, Appendix S (0.1g + spectral shape at foundation)

Table 10-2 Elements of SSI and structure response in the United States and Japan in operating and new reactors

Elements of the Process	Japan Pre-2006 Design	Japan Post-2006 Design	U.S. Pre-2007 Design	U.S. Post-2007 Design
Input Motion	Derived from S_1 and S_2 specified on Base Stratum (with 1D wave propagation theory applied to propagate motion to other locations, if necessary); Recorded time histories or artificially generated; Horizontal component	Assume to be derived from S_s and S_d specified on Base Stratum (with 1D wave theory applied to propagate motion to other locations, if necessary); Assume recorded time histories or artificially generated; Horizontal and vertical components.	Defined TOG or on outcrop of competent material (1D-wave propagation theory to propagate to other locations, if necessary); three components (two horizontal and the vertical)	Defined TOG, FIRS, or other locations taking into account site response analyses from hard rock to location of interest; three components (two horizontal and the vertical).
Soil Profiles	Equivalent Linear Best Estimate	Assume Equivalent Linear Best Estimate	Three soil profiles (equivalent linear) BE, UB =(1+COV) BE, LB=BE/(1+COV) Envelope results	Three soil profiles (equivalent linear) - BE, UB=(1+COV)BE, LB=BE/(1+COV) Envelope results
Structure Damping	Specified – relatively low	To be determined	RG 1.61	RG 1.61
Nonlinear Behavior of Soil-Structure Interface	Sliding and Uplift Analyzed for Class A_s Structures for S_2	To be determined	Check for Sliding, Uplift, Separation (Separation modeled equivalent linearly)	Check for Sliding, Uplift, Separation (Separation modeled equivalent linearly)
Nonlinear Behavior of Structure	Nonlinear Structure Behavior Analyzed for Class A_s Structures for S_2	Nonlinear Structure Behavior to be Analyzed for Class S Structures for S_s	Modeled as Equivalent Linear	Modeled as Equivalent Linear
Structure Responses	Dynamic Forces/Moments to be Combined with Static Vertical Component; In-structure response spectra (ISRS) peak broadened ± 10% and smoothed (horizontal only).	Dynamic Forces/Moments for Horizontal and Vertical Input Motions. ISRS to be determined.	Envelope of three soil cases; Dynamic Forces/Moments for Two Horizontal and the Vertical Input Motions combined by Algebraic Sum, SRSS,100/40/40, others; ISRS envelope of three soil cases, peak broadened +/- 15%, smoothed and valleys filled in.	Envelope of three soil cases; Dynamic Forces/Moments for Two Horizontal and the Vertical Input Motions combined by Algebraic Sum, SRSS, 100/40/40, others; ISRS envelope of three soil cases, peak broadened ±15%, smoothed and valleys filled in.

Table 10-3 Seismic classifications in the United States and Japan in operating and new reactors

	Japan Pre-2006	Japan Post-2006	U.S. Pre-2007 Design	U.S. Post-2007 Design
Seismic Classification	Classes A_s, A, B and C Classes A_s and A designed to maximum of S_1 and static Class A_s evaluated to S_2 Classes B and C designed to static loads	Classes S, B, and C; Class S is combination of Classes A_s and A SSCs and some Class B equipment; Class S SSCs elastically designed to maximum of EDGM S_d and static; Class S evaluated to S_s; Classes B and C unchanged	Seismic Categories – I and –II and non-seismic; SC–I designed to OBE and SSE; SC–II evaluated to assure no failure, which would cause failure to SC–I SSCs; Non-seismic SSCs designed to industrial standards	Seismic Categories –I and –II and non-seismic; SC–I designed to SSE; SC–II evaluated to assure no failure, which would cause failure to SC–I SSCs; Non-seismic SSCs designed to industrial standards

Table 10-4 Structure design in the United States and Japan in operating and new reactors

	Japan Pre-2006	Japan Post-2006	U.S. Pre-2007 Design	U.S. Post-2007 Design
Structure Loads	See Table 6-1	See Table 6-2	Envelope of three soil cases; Dynamic Forces/Moments for Two Horizontal and the Vertical Input Motions combined by Algebraic Sum, SRSS, 100/40/40, others	Envelope of three soil cases; Dynamic Forces/Moments for Two Horizontal and the Vertical Input Motions combined by Algebraic Sum, SRSS, 100/40/40, others
Structure Design	S_1* - Suitable Standard Building Code S_2 – demonstrate margin to ultimate strength for Class A_s SSCs	S_d - Suitable Standard Building Code S_s – demonstrate margin to ultimate strength for Class S SSCs	NRC SRP and RGs, Industry Codes	NRC SRP and RGs, Industry Codes

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APPENDIX A
SUMMARY OF KEY LEGAL REQUIREMENTS AND GUIDELINES IN JAPAN

Table A-1 Legal requirements and guidelines in Japan in place at the time of the 2011 Tōhoku earthquake

Governing Organization	Codes or Guides	Provisions
Nuclear Safety Commission of Japan (NSC)	**"Regulatory Guide for Reviewing Safety Design of Light Water Nuclear Power Reactor Facilities"**	**Guidelines 34 to 40: "Safety Protection System"** ***Guideline 34: Redundancy of the safety protection system*** The safety protection system shall be designed to be redundant such that it does not lose its safety protection function even when a malfunction of a piece of equipment or subsystem that makes up the safety protection system occurs, or when a piece of equipment or a subsystem is removed from service. ***Guideline 35: Independence of the safety protection system*** The safety protection system shall be designed so that elements that make up the system are separated from one another. Independence of the elements should be considered to the extent practicable so the safety protection system does not lose its safety protection function at the time of normal operation, maintenance, tests, and abnormal conditions. ***Guideline 36: Function of safety protection system at the time of a transient*** The safety protection system shall be designed to detect an abnormal condition should an abnormal transient occur during normal operation. The safety protection system should automatically activate appropriate systems including the reactor shutdown system, and the allowable design limit of the fuel should not be exceeded. ***Guideline 37: Function of safety protection system at the time of an accident*** The safety protection system shall be designed to detect an abnormal condition at the time of an accident and to automatically activate reactor shutdown system and engineered safety features.

Table A-1 Legal requirements and guidelines in Japan in place at the time of the 2011 Tōhoku earthquake (continued)

Governing Organization	Codes or Guides	Provisions
Nuclear Safety Commission of Japan (NSC)	“Regulatory Guide for Reviewing Safety Design of Light Water Nuclear Power Reactor Facilities”	***Guideline 38: Function of safety protection system at the time of a malfunction*** The safety protection system shall be designed so that the reactor is brought to a safe condition even if loss of power, system cutoff, or other unfavorable conditions occur. ***Guideline 39: Separation of safety protection system from measurement and control systems*** The safety protection system shall be designed to be functionally independent from measurement and control systems so as not to lose inadvertently its safety protection function due to the measurement and control systems if it shares a part of the system with measurement and control systems. ***Guideline 40: Testability of safety protection system*** The safety protection system shall be designed so that it can be tested at regular intervals during reactor operation and so that each subsystem can be tested independently to check that integrity and redundancy are maintained.
Ministry of Economy, Trade, and Industry (METI)	METI Ordinance Number 62 (before Jan 2006 revision)	**Article 22: "Emergency Shutdown Features"** At a nuclear power plant (NPP), features with the following functions shall be installed: • Systems and components to detect, without fail, the possibility that the reactor may be unable to continue to operate safely due to significant increase of thermal output of the reactor, significant decrease of the removal capability of the heat generated in the reactor pressure vessel, or occurrence of earthquake ground motions • Systems and components needed to shut down the reactor operation automatically and promptly such that the allowable fuel damage limit is never exceeded **Interpretation** In the “Interpretation” of Article 22, it was also stated that one of the conditions that may adversely affect safe operation of the reactor is “excessive earthquake acceleration.” So it was believed that it was legally required to scram the reactor with a signal from the seismic trigger (acceleration) in Japan.

Table A-1 Legal requirements and guidelines in Japan in place at the time of the 2011 Tōhoku earthquake (continued)

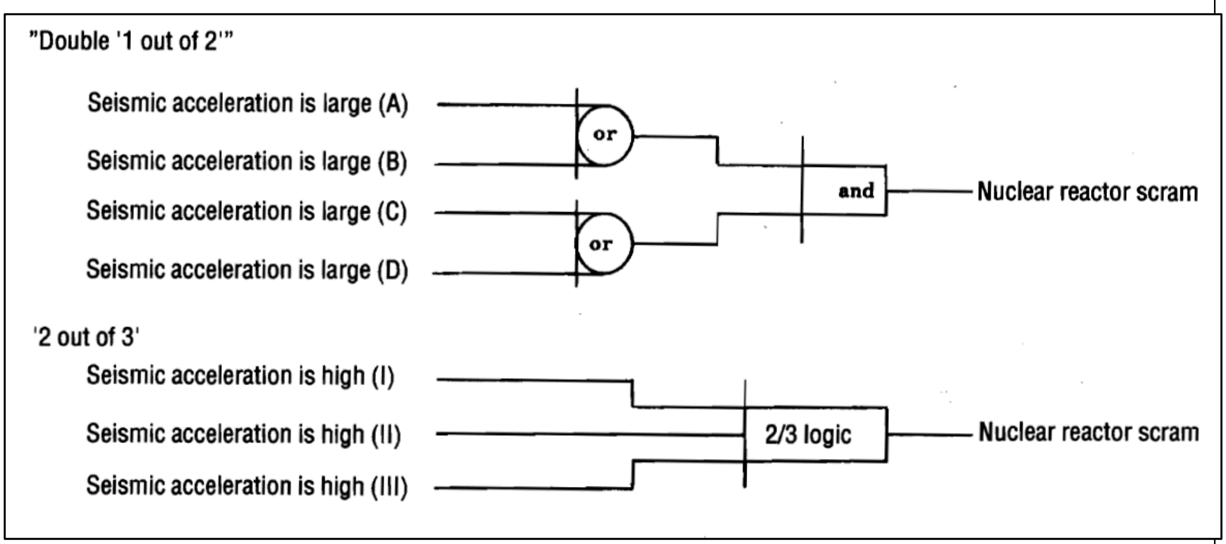
Governing Organization	Codes or Guides	Provisions
Ministry of Economy, Trade, and Industry (METI)	METI Ordinance Number 62 (after Jan 2006 revision)	**Article 22: "Safety Protection Features"** An NPP shall design the safety protection features to address the criteria below. 1. In the event of an abnormal transient during operation or a disruption of the reactor operation due to an occurrence of earthquake, the safety protection features (in cooperation with the reactor shutdown systems and the engineered safety features) shall function in order to keep the reactor below the allowable fuel damage limit. 2. Equipment or subsystem that make up the system are to be redundant so that the safety protection system does not lose its safety protection function even when a single malfunction to the equipment or subsystem might occur, or when equipment or subsystem are removed from service. (Criteria 3 through 7 are not applicable here) **Interpretation** In the "Interpretation" of Article 22, it is stated that the function of the safety protection features shall be confirmed to meet the specifications and safety evaluation conditions in the established licensing documents. Additionally, "Double 1 out of 2" logic is described as one example to meet the requirement of Item 2. (See the figure at the end of this table for an example of Double 1 out of 2 logic)

Table A-1 Legal requirements and guidelines in Japan in place at the time of the 2011 Tōhoku earthquake (continued)

Governing Organization	Codes or Guides	Provisions
Japan Electric Association (JEA)	JEAG 4601-1987 (Endorsed by the Nuclear and Industrial Safety Agency of Japan)	**Attachment 3: "Earthquake detecting equipment"** In an NPP, if the nuclear reactor cannot continue to operate safely, this state must be detected by a safety protection system and the operation of the nuclear reactor must be shut down automatically. During an earthquake, if certain abnormal phenomena caused by the earthquake take place such that the nuclear reactor cannot continue to be operated safely, the nuclear reactor must be shut down by this safety protection system. In the case where the seismic motion is greater than the design seismic motion for the safety-related equipment, the nuclear reaction should be shut down in order to effectively ensure the safety of the NPP. For this purpose, an earthquake motion-detecting system that can shut down the nuclear reactor (if the earthquake motion is above a certain shaking level) must be installed in the NPP as a safety protection system. For the earthquake-motion detecting system, the location of the seismometer (also called "seismic trigger" below), the SCRAM triggering level, and the earthquake SCRAM logic circuit are as follows: **(1) Location of seismic trigger** The location of the seismic trigger of the earthquake-motion detecting system should be determined with consideration of the objective for which the seismic motion is to be detected. The selected location should be easily accessible for maintenance and inspection activities. The location should be chosen to ensure, with high reliability, that the motions observed by the monitoring equipment could be effectively used to meet the objective. As a result, the seismic trigger should be located on the same floor as the equipment important to safety that it is intended to protect. Specifically, in a building that contains equipment important to safety, the seismic trigger is oriented in the horizontal direction and located on the lowest story of the building in order to effectively detect the seismic motion input to the building. In some cases, a seismic trigger, oriented in the horizontal direction, is also located on a "typical" floor on an upper floor, and a seismic trigger oriented in the vertical direction is also located on a "typical" floor. **(2) Earthquake SCRAM acceleration level**

Table A-1 Legal requirements and guidelines in Japan in place at the time of the 2011 Tōhoku earthquake (continued)

Governing Organization	Codes or Guides	Provisions
		The SCRAM trigger level is the predetermined acceleration value at which the nuclear reactor is shut down automatically by the earthquake detection system. To meet its stated purpose, the system must be able to reliably detect seismic accelerations near or exceeding the amplitude of the S_1 seismic design motion because this is seismic design level of interest to the equipment important to safety. **(3) Earthquake SCRAM logic circuit** The earthquake-motion detection device is a safety protection system that can automatically stop the nuclear reactor quickly after an earthquake takes place. Based on the basic design guideline of the safety protection system, the earthquake scram logic circuit may have the form of "double 1 out of 2" or the form of "2 out of 3," as shown in the figure below.



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10. SUPPLEMENTARY NOTES

11. ABSTRACT (200 words or less)

Over the years, a number of nuclear power plants (NPPs) in Japan have experienced earthquake shaking and some have experienced shaking in multiple earthquakes. The U.S. Nuclear Regulatory Commission has identified a need to better understand the seismic performance of Japanese NPPs and to determine if any important lessons should be applied to NPPs in the United States (U.S.). Meeting that goal requires an understanding of the design criteria used in Japan, and the differences between the practices employed in the two countries.

This report provides information on current and past U.S. and Japanese seismic design standards, calculational methods, and load combinations used for the design of new and currently operating NPPs up to approximately 2012. This report documents the relative conservatism of the U.S. and Japan seismic analysis, design, and qualification inputs and processes. Both countries generally employ techniques that provide significant margin against the earthquake shaking levels used for design. This report covers various timeframes of interest for both countries.

This report provides information in the areas of seismic hazard assessment, classification categories, soil structure interaction analyses, structural design, subsystem analysis and design, beyond design basis events, and seismic instrumentation. The report provides an assessment of the conservatisms in both the U.S. and Japanese approaches.

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