A SUMMARY OF THE HISTORICAL DEVELOPMENT OF THE SEISMIC DESIGN OF NUCLEAR POWER PLANTS IN JAPAN AND THE U.S.

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ORIGINAL NPP JAPANESE SEISMIC LOAD REQUIREMENTS BEFORE 1965

Seismic NPP Requirements in Japan were originally developed for:

- English Gas Cooled Reactors, 1957 – Japanese Building Code of the time
- Westinghouse PWR – Mihama Unit One, 1964 – 0.29g pga
- G.E. BWR – Tsuruga and Fukushima Unit 1, 1965 – 0.24g pga and 0.17g; respectively

ORIGINAL NPP U.S. REQUIREMENTS SEISMIC LOAD REQUIREMENTS BEFORE 1965

Seismic NPP Requirements in the U.S. were originally defined for:

- U.S. G.E. BWR – Humbolt Bay, 1962 – a single 0.25g Design Basis Earthquake

- U.S. PWR – Connecticut Yankee, 1964 – 0.03g (evaluated later for 0.17g pga) Originally designed based on Building Code Requirements with Nuclear Safety Related buildings modeled as fundamental 1 horizontal dynamic degree of freedom system with UBC static analysis used for design of safety related systems and components.

Seismic Design Requirements

There are 3 areas that require definition to be used in seismic design of a NPP:

- Quantification of Seismic Load and other Applicable Loads and Specification of their Load Combinations to be considered in design
- Analysis Methods and Procedures (necessary to convert input earthquake motions normally expressed in the form of acceleration to resultant seismic forces, moments or stresses, fms in structures, systems and components, SSC
- Codes or standards acceptance criteria used to evaluate resultant forces, moments or stresses in nuclear safety SSC to determine design adequacy.

Seismic Design Requirements

To determine the loads and load combinations to be considered in design, it is necessary to safety/seismic classify those SSC to which each load and load combination is applicable.

The first and third design requirements are directly controlled by national construction codes and standards while the second is associated with analytical procedures is controlled by a combination of the education and experience of the design engineer and codes and standards. In many instances the applicable codes or standards require a peer review and approval of the design and analysis by a licensed or otherwise qualified professional engineer.

JAPANESE SEISMIC LOAD ACCEPTANCE REQUIREMENTS AFTER 1970

- Two dynamic earthquake criteria for A and As Safety Classes.
  - Static Building Code required for B and C Safety Class SSC. Building Code short term load acceptance criteria applied to Safety Class A.
  - Ultimate Strength applied to buildings and 2 x yield stress applied to pressure retaining components for load combinations which included S_2.

U.S. SEISMIC LOAD ACCEPTANCE REQUIREMENTS AFTER 1970

- Safe Shutdown Earthquake:
  - Largest recorded in history within a 200 mile radius moved to site proximity or largest recorded fault motion for capable faults considered. Allowed stresses for buildings increased to yield and for pressure retaining mechanical distribution systems to 2 x yield analyzed elastically.

- Operation Basis Earthquake
  - This earthquake is defined equal to one-half the SSE and Structures, Systems and Components with normal allowable (i.e. 2/3 yield acceptance criteria).

  - Both SSE and OBE applied to all Seismic Category I SSC (i.e. associated with reactor safety).

TABLE 1 Typical Historical Levels of NPP Design Basis Earthquakes in Japan

Design Basis Earthquake	Design Basis Seismic Coefficient (Static) Times Gravity	Dynamic ZPGA[(1)] Gal	Applicable to Safety Class SSC
S_2 (margin check upper bound)	0.9	240 Typically[(2)] 480 or 600(=1.5S_1)	A_S
S_1 (design basis historical max.)	0.6	180, 300 or 450	A
s_o (operational) once per 2~10 years	----	50~60	All
s_B	0.3	90~200 (=1/2 S_1)	B
s_C	0.2	90~100 (=1/3 S1)	C

(1) ZPGA* is “Dynamic Design Basis Peak Acceleration”

(2) Depending on the plant site region historical earthquake intensity

TABLE 2 U.S. Correlation of Seismic Categories of ANS 2.26 and ASCE-43 with Existing U.S. DOE and U.S. NRC Seismic Classification of SSC

Safety Categories	DOE Seismic Classification	NRC Seismic Classifications at NPP
SDC-5	None	Seismic Category 1
SDC-4	Safety	Seismic Category 1
SDC-3	Safety Significant with a Confinement Function	RW-IIa
SDC-2	Safety Significant and Important to Safety and Defense in Depth	RW-IIb
SDC-1	Not Important to Safety	RW-IIc

Notes:

1. SDC-5, which is equivalent to S_2 a 1.5 factor, would typically be applicable to the site earthquake as a foundation of the slope of the seismic hazard curve and the safety goal.
2. For SD-4 a 1.o factor would typically be used.

Table 3a Japanese NPP Aseismic Design Data for NPP Before 1973

Name of Plant	Location	Seismic Data		
		A	B	C
Onagawa	Miyagi	0.48	0.24	250
Fukishima (No. 1, 2 3)	Fukishima	0.48	0.24	180
Hamaoka	Shizuoka	0.48	0.24	300
Mihama	Fukai	0.48	0.24	300
Takahama	Fukai	0.48	0.24	270
Shimane	Shimane	0.48	0.24	200
Tokai	Ibaragi	0.48	0.24	--
Tsuruga	Fukai	0.48	0.24	250
Genkai	Saga	0.48	0.24	150
Fugen Atr	Fuki	0.48	0.24	250

Notations:

A. Design seismic coefficient static horizontal, g (C_o x 3 x 0.8)
B. Design seismic coefficient static vertical, g (C_o x 1.5 x 0.8)
C. Design seismic coefficient dynamic horizontal, gal., S_1

C_o Seismic coefficient; for most of the cases C_o = 0.20

*Personal communication between K. Muto and D.S. Mehta, P.T. Kuo and A.A. Vizzi, November 1972.

Table 3b Typical U.S. NPP Seismic Design Data for NPP Before 1973

Name of Plant	Location	Seismic Data		
		A	B	D
Diablo Canyon(1)	California	0.4	0.2	0.67
San Onofre, Unit 2	California	0.67	0.33	0.67
Trojan	Oregon	0.25	0.15	0.67
D.C. Cook	Michigan	0.20	0.10	0.67
Davis Besse	Ohio	0.15	0.08	0.67
Indian Point	New York	0.15	0.10	0.67
Duwane Arnold	Iowa	0.12	0.06	0.80
Turkey Point	Florida	0.15	0.05	0.67
St. Lucie	Florida	0.10	0.05	0.67

Notations:

(1) Later changed to 0.67g and 0.33g

D = Ratio of Vertical to Horizontal PGA

Historical Development of Analytical Models

Prior to 1965 static seismic lateral loads on structures were defined on the basis of the fundamental frequency of the structure and an assumed flexibility based on a $(k/m)^{1/2}$ relationship with m being the mass of the structure and k its axial and bending stiffness plus if significant, its shear stiffness.

HISTORICAL DEVELOPMENT OF ANALYTICAL MODELING BETWEEN 1965 AND 1967

- Multi-degrees of freedom (3 mass) model using Housner response spectra as input, for containment structure including in the U.S. springs representing Soil Structure Interaction where plant founded on soil media. Japanese safety related structures founded on rock.

- Other safety related structures were designed using one degree of freedom equivalent static analysis treating the structure as single degree of freedom systems

- System and components modeled statically with peak of Hounser Spectra shape normalized to high frequency (20 Hz) floor acceleration. The concept of ISRS did not appear until after 1967.

HISTORICAL DEVELOPMENT OF ANALYTICAL MODELS BETWEEN 1967 AND 1970

After 1967 computer programs became available in the nuclear industry which permitted multi-degree of freedom modeling of nuclear safety related buildings subjected response spectrum modal analysis.

Equivalent static analysis equal initially to peak of ground response spectra normalized to building floor acceleration and then to the floor or in-structure response spectra when they became available by 1970 were used to analyze distribution systems and components.

U.S. Increase of Building Floor Motion Acceleration as a Function of the Elevation of the Floor Above the Base

Shear Wall	1 + 2 x /h
Concrete Frame	1 + 3 x /h
Steel Frame	1 + 4 x /h

where:

x = floor height of building

h = total height of building

Historical Development of Analytical Models After 1970

While commercial computer programs became available about 1967 to analyze dynamic multi-degrees of freedom system and were used with either real and artificial acceleration time histories to generate in-structure or floor response spectrum, it was not until the early 1980's that computer programs became available for direct generation of floor or in-structure response spectra as a function of the ground spectra and dynamic characteristics of the building structure without the necessary generation or use of an input time history. After 1970 in the U.S., the Housner Response Spectra was replaced by the R.G. 1.60 mean plus 1 seismic response spectra.

Peak spectral ground or floor or in-structure Response Spectra ratio amplification factors:

Damping	Acceleration	
	Ground	Floor
2%	4	6.0
5%	3.0	4.0
7%	2.0	3.0

Based on R.G. 1.60 shaped spectra

Historical Development of Analytical Models After 1975

By 1975 it began to be recognized that for unsymmetrical buildings the applied motion in one orthogonal direction could cause significant response motion in a different orthogonal direction due to coupling. As a result, the dynamic one dimensional beam stick beam model was replaced with 6 degrees of freedom dynamic beam-column stick consisting of one-dimensional finite elements. As a result, the seismic response the model had not only to consider summation of modal responses and primary direction al responses, but also the summation of directional responses where the application of an earthquake in one direction produced responses in all three orthogonal directions. This was accomplished in general by separating at a particular building elevation the center of mass from the center of shear in the model. The actual combination of seismic induced stresses or forces and moments in individual building members or sections was by either SRSS or ABS for clearly shaped modes and either SRSS or 100 40 40 for directions.

Historical Development of Analytical Models for Buildings on Soil

The initial modeling of structures was to assume a rigid building foundation. In Japan this modeling continued as a result of using site with rock foundations. Soil structure interaction for Seismic Category I building structures was started in the U.S. in the late 1960's by use of a one-dimensional translational and rotational springs representing the building foundation stiffness as a function of its foundation soil shear wave velocity and density and the use of experimentally determined levels of radiation damping in the foundation soil. This type of SSI analysis gave way to use of a continuum method which was used as a soil impedance function method in the 1980's. Since the 1990's finite element modeling of the foundation media, coupled with the building model, has been used with computer programs such as SASSI or more generalized finite element programs such as ANSYS. The SASSI program which solved the dynamic program in the frequency rather than the time domain is a linear analysis program which does not treat non-linear geometry effects such as lift-off or variable stiffness due to foundation or building material non-linearity. ANSYS is a computer program that does have the non-linear analysis capability, but at this time SSI models because of their size and complexity solved dynamically and non-linearly are not considered practical.

Current Analytical Models

It has become possible for relatively simple and small building to be modeled dynamically with either two or three-dimensional finite elements and determine directly the seismic stress, forces and moments in individual beam or column members or concrete floor or wall sections.

A three-dimensional column stick model is still used to determine in-structure response spectra because the 3D finite element model tends to generate localized response spectra which are not consistent with the overall response of the elevation level being considered.

Current Analytical Models

For large or complex building models a two-step process is used where a simplified dynamic building model, either stick or a reduced finite element model is used to generate equivalent static input typically in the form of bubble plots to a detailed static finite element model of the building.

Japanese Acceptance Criteria for Buildings

Current Japanese Building Load Combinations Acceptance Criteria and Thermal Stress

Allowable Stress Rate	Load Combinations	Thermal Stress
Long-term	1 $(D + L) + O + T_1$	Reduced to ½
Short-term	2 $(D + L) + O + T_1 + K_1$ 3 $(D + L) + LO + T_2$	Reduced to 1/3
Ultimate	4 $(D + L) + O + K_2$ 5 $(D + L) + LO + K_1$	The thermal stress is not considered

Symbols:
D : dead load
L: live load
O: operation load
LO: load in L-accident
T_1: temperature load in operation
T_2: temperature load L-accident
K_1: seismic force due to S_1 earthquake
K_2: seismic force due to S_2 earthquake

Allowables defined in Japan Building Code for long and short-term loading. The ultimate capacities are based on strength design procedures.

U.S. Acceptance Criteria for Buildings

U.S. Design Basis Loads and Load Combinations for Design of Nuclear Safety Related Steel and Concrete Structures

	AISC N690-06 – Steel Structures		**ACI-349-06 - Concrete Structures**
	Normal Load Combinations		**Normal Load Combinations**
1.	$1.4 (D + R_o) + T_o + C_{cr}$	1.	$1.4 (D + F + R_o) + T_o$
2.	$1.2 (D + R_o + T_o) + 1.6L + 1.4C + 0.5(L_r \text{ or } S \text{ or } R)$	2.	$1.2 (D + F + T_o + R_o) + 1.6 (L + H) + 1.4C_{cr} + 0.5 (L_r \text{ or } S \text{ or } R)$
3.	$1.2 (D + R_o + T_o) + 1.6 (L_r \text{ or } S \text{ or } R) + 0.8L + 1.4C$	3.	$1.2 (D + F + T_o + R_o) + 0.8 (L + H) + 1.4 C_{cr} + 1.6 (L_r \text{ or } S \text{ or } R)$
	Severe Environmental Load Combinations		**Severe Environmental Load Combinations**
4.	$1.2 (D + R_o) + 1.6W + 0.8L + C + 0.5 (L_r \text{ or } S \text{ or } R) + T_o$	4.	
5.	$1.2 (D + R_o) + 1.6E_o + 0.8L + C + 0.2 (L_r \text{ or } S \text{ or } R) + T_o$	5.	$1.2 (D + F + R_o) + 1.6 (L + H + E_o)$ $1.2 (D + F + R_o) + 1.6 (L + H + W)$
	Extreme Environmental and Abnormal Load Combinations		**Extreme Environmental and Abnormal Load Combinations**
6.	$D + 0.8L + C + T_o + R_o + E_S$	6.	$1.0 (D + F + C_{cr} + H + T_a + R_a + E_s) + 0.8L$
7.	$D + 0.8L + T_o + R_o + W_t$	7.	$1.0 (D + F + C_{cr} + H + T_a + R_a + W_t) + 0.8L$
8.	$D + 0.8L + C \text{ or } 1.2P_a + R_a + T_a$	8.	$1.0 (D + F + C_{cr} + H + T_a + R_a) + 1.2 P_a + 0.8L$
9.	$D + 0.8L + (P_a + R_a + T_a) + (Y_r + Y_j + Y_m) + 0.7E_s$	9.	$1.0 (D + F + C_{cr} + H + T_a + R_a + P_a + Y_r + Y_j + Y_m) + 0.8L$

1. See ACI 349 Code and N-690 Standards for notes to table.
2. Normal loads with load factor of 1.0 have an expectation of 1.0/year
3. Severe Environmental loads with load factor of 1.0 have a mean expectation of 10^{-2}/year
4. Extreme Environmental loads have a mean expectation of 10^{-4}/year for SDC-5 and 4 and $4x10^{-4}$/year for SDC-3 for DOE

Significant Differences Between Japan and U.S. Seismic Design Criteria

Design	Japan	U.S.
Seismic Ground Motion	Applied at Rock Interface	Applied at Free-Field Ground Surface (until recently)
Vertical Seismic Motion – Assumed Constant	Vertical Seismic Acceleration Taken as a Constant (until recently)	Vertical Response Spectra Was Used
Earthquake Design Basis Ground Accelerations at the Site	Based on Historical Earthquake Experience	Based on Mean 10^{-4}/year probability of exceedence (since mid 1990's)
S_1 and S_2 Earthquake	Applied to Different A or A_s SSC	Both SSE and OBE Applied to Same Seismic Category I SSC
Damping Assumed for Piping	Two to Three Percent of Critical	Four Percent of Critical
Containment	Seismic Load Not Combined with LOCA Load	SSE Seismic Load Combined with LOCA Load; Load Factor = 1.0
Rock	Typically Defined at > 400 m/sec Shear Wave Velocity Based on Japanese Earthquake Frequency Characteristics < 10 Hz	Defined at > 700 m/sec Shear Wave Velocity Based on U.S. Earthquake Frequency Characteristics in East > 10 Hz

Some Significant Seismic Design Conservatisms

- "Worst case enveloping" approach is used in:
  - Design basis control motions
  - Floor response spectra
  - Load combination
- Lower bound minimum values are specified for:
  - Damping values
  - Material properties
- Linear elastic analysis for Design Basis load without taking credit for:
  - Soil nonlinear behavior
  - Elastic-plastic response of ductile materials (Energy Method not allowed)

Approximate Percent

Seismic Direct Cost of Plant Plus Engineering For a

1200 MWe LWR equal to a 0.2g pga Design Basis (See Fig. 1)

Item	Percent of Direct Plant Costs
Seismology & Siting	6.2
Foundation	26.4
Building Structure	2.5
Auxiliary Components	11.9
NSSS	3.8
Distribution Systems Analysis and Supports	48.7
Turbine Hall	0.6

Total direct cost of plant taken equal to $\$2.7 \times 10^9$

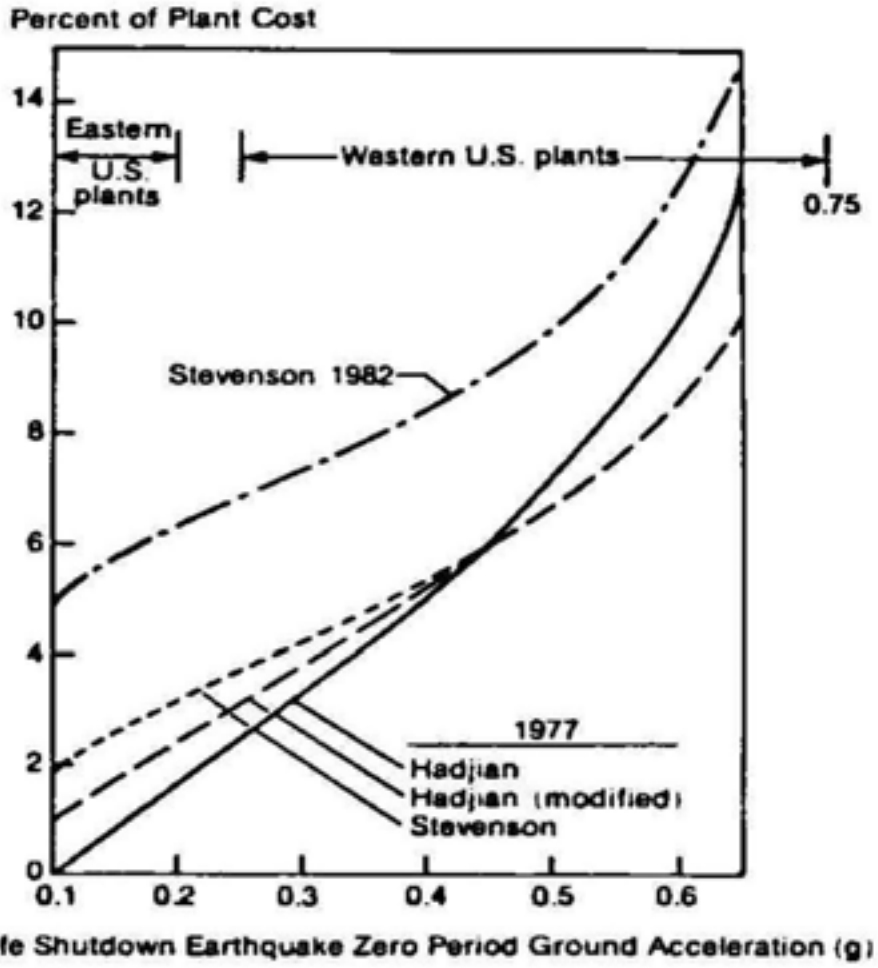

Figure 1

COMPARISON OF BEST ESTIMATE SEISMIC COST INCREASE

Conclusion

GENERAL:

Seismic design requirements in both Japan and the U.S. have had a significant effect on safety related NPP design ranging in the U.S. from about 6 to 15 percent of plant direct costs with an individual plant costs as high as 22 percent (i.e. Diablo Canyon).

In the area of building design the seismic design requirement has lead to shorter and wide based building layouts associated with a nuclear island with some increase in reinforcing steel in concrete structures to carry in-plane seismic shear and overturning. It has also lead to a significant increase in the engineering effort required to analyze and design such structures.

Conclusion

BUILDINGS:

For safety related buildings, in particular containment embedment's into the foundation media, is generally a layout requirement resulting in a major effort to provide a suitable foundation media requiring a rock like foundation media or large quantities of engineered backfill.

Conclusion

MECHANICAL:

It is in mechanical and electrical distribution system design where seismic requirements have their greatest impact. Lateral supports required to carry seismic loads are typically spaced every 2 to 4 times deadweight support spacing as a function of design basis seismic lateral acceleration. In elevated temperature systems many such supports are required to be active in nature where they move when subjected to slowly applied thermal loads yet lock up when subjected to suddenly applied seismic loads. The lateral support of distribution systems is by far the largest increment of seismic induced design effect and hardware costs.

Conclusion

POWER INSTRUMENTATION AND CONTROL:

For electrical and instrumentation and control devices and components it is usually necessary to provide seismic qualification by dynamic earthquake simulated shake table testing which adds significant cost of such devices or components.

Conclusion

The analytical and testing methods associated with seismic design have required the continuous development of more and more sophisticated and rigorous qualification procedures, as well as a significant increase in the perceived level of applicable seismic loading over the past 40 years. Current seismic evaluations in Japan are for an S_s response spectrum equal to 1.0g pga. In the U.S. a Beyond Design Basis Earthquake equal to 1.67 times the SSE to demonstrate no failure is required. It is hoped that the seismic design requirements applied to NPP has finally reached an acceptable level of maturity and the constant major changes in methodology and qualification procedures may finally have stabilized in order to allow nuclear power to continue to be a major economical resource for generation of electrical energy.