HISTORICAL INTERNATIONAL DEVELOPMENT OF SEISMIC DESIGN AND ANALYSIS OF NUCLEAR POWER PLANT STRUCTURES, SYSTEMS AND COMPONENTS OVER THE LAST 60 YEARS

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INTRODUCTION

The commercial nuclear power era began in the U.S. in 1957 with the completion of the Shippingport PWR type reactor plant. Earlier power generation from a nuclear reactor occurred from government reactors in service in the U.S. and other countries.

At the beginning of the Nuclear Power Plant, NPP era four countries developed commercial nuclear power plant systems and the major developers, producers and constructors of NPP between 1957-1965:

- Canada
- Great Britain
- Soviet Union (Russia after 1990)
- United States

U.S. Influence

Starting about 1965 they were joined by:

- Japan
- France
- W. Germany
- Belgium
- Holland
- Spain
- Sweden

Using basic reactor types and influenced by licensing and reactor design and safety criteria developed in the U.S.

Soviet Union Influence

The eastern European countries of the

- Soviet Union
- E. Germany
- Czechoslovakia
- Hungary
- Bulgaria

Used primarily VVER type Pressurized Water Reactor, PWR developed in the Soviet Union.

Reactor Types – U.S.

- Pressurized Water
- Boiling Water Reactors

Employed both light-water cooled and moderated slightly enriched U235 reactor cores.

Reactor Types – Canada & Britian

- Canada: heavy-water moderated and light-water cooled natural uranium reactor cores.

- Britain: carbon dioxide cooled graphite moderated natural uranium cores.

Commercial Power Reactors – U.S. and Canada

- High energy density cores capable of core melt with the loss of forced core cooling as the result of decay heat

- Fitted with containment structures surrounding the reactor coolant system which included
  - Reactor
  - Steam generator
  - Reactor coolant pump pressurizer in a PWR
  - Reactor and reactor coolant pumps and steam lines in a BWR.

Commercial Power Reactors - Britain

- Gas-cooled reactors with their lower core energy density
- Not capable of core melt resulting from decay heat as a result of loss of forced cooling
- Not fitted with containment structures and systems.

Commercial Power Reactors - Russia

- Predominately of the PWR type
- Capable of core decay heat melting
- Not fitted with containment structures and systems until the late 1980's.

Containment
Versus
Confinement

Confinement

A barrier and its associated systems placed around a volume containing toxic materials to prevent their uncontrolled release. Criteria for design:

- **Barrier system**: allows inelastic behavior (e.g. Limit State C or D) under all design bias loading conditions
- **Ventilation**: limited leakage may be allowed
- **Confinement volume**: may be designed to remain under slight negative pressure (up to 2 to 4 cm water gauge) under normal operating conditions

Containment

A leak-tight barrier and its associated systems enclosing a volume containing highly radioactive material. Criteria for design:

- Leak-tightness must be maintained under all design basis loading conditions,
- System must remain elastic (Limit State D) under all design loading conditions except under impact and impulse loads where limited inelastic behavior may be permitted,
- Constructed containment subjected to Structural Integrity Test (SIT) and Integrated Leak Rate Test (ILRT) to demonstrate leak tight integrity.

Containment Design

PWR: contain the reactor coolant system inventory

BWR: contain both primary reactor coolant system and secondary steam supply system out to the containment isolation valve

Export of Reactor Systems

Great Britain effectively has not exported its reactor type. Canada exported its reactor in the 1970's principally to Argentina, Korea and Romania. Korea has also installed U.S. type pressurized water and Canada heavy water moderated NPP. Germany and France have also exported commercial power reactors.

It only has been in the last 20 years or so that Japan, Korea and India have developed their own reactor systems primarily of the light-water cooled reactor type. In the past 15 years China has been a major importer of pressurized water reactor nuclear power plants initially from France followed by Russia and most recently by the U.S. China has also developed and installed its own PWR type reactor system and has exported a PWR to India.

Seismic Background

The seismic design of commercial Nuclear Power Plants, NPP have undergone drastic changes over the 55 years since their inception. The initial design of NPP followed the then available national building codes of the late 1950's, which were based on static analysis for earthquakes with an approximate mean 100 year return period (10^{-2}/yr probability of exceedence). The seismic loads were statically applied laterally with only a 0.05g acceleration in low seismic intensity areas, 0.10g in moderate seismic intensity areas and 0.2g in areas of high seismic intensity. The explicit application of seismic loads was limited to building structures.

Seismic Background

The analytical procedures used to develop forces and moments and resultant seismic stresses in individual building members were limit to hand calculations. The acceptance criteria was typically limited to a one-third increase in normal allowable stresses when seismic stresses were combined with normal dead and live load stresses where normal allowable stresses were between 0.6 and 0.67 times specified minimum yield.

Seismic Design Activities

There are three activities associated with the design of civil, mechanical, electrical and instrumentation and control structures, systems and components to resist seismic effects.

- Quantification of Seismic Load and other concurrent 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, fms in nuclear safety SSC to determine design adequacy.

Seismic Design Activities

The first and third design activities are defined by applicable national codes, standards and regulatory requirements. The second activity is primarily a function of the experience and training of the engineer(s) performing the analysis and their understanding of the state-of-the-art of dynamic analysis of structural systems by finite element methods to include also any specified analytical requirements specified by applicable national design or construction codes, standards and regulatory authorities.

Seismic Design Load Requirements

In the world today there are two basic procedures used to define seismic loads. The first procedure is to use a probabilistic approach where in the U.S., Canada and Russia the Design Basis Earthquake is defined at the mean 10^{-4}/yr probability of exceedence. In Germany, France, Japan, Korea and India the Design Basis Earthquake is defined deterministically based on maximum historical seismic intensity scales such as the MSK, JMA or the EMS damage scales. In general, the seismic loading phenomena is expressed as seismic motion induced inertial accelerations when applied to the mass distribution of the SSC resulting in forces to be applied to structural models of the SSC to determine member or component stresses.

SEISMIC LOADS

1957 – 1965 Era

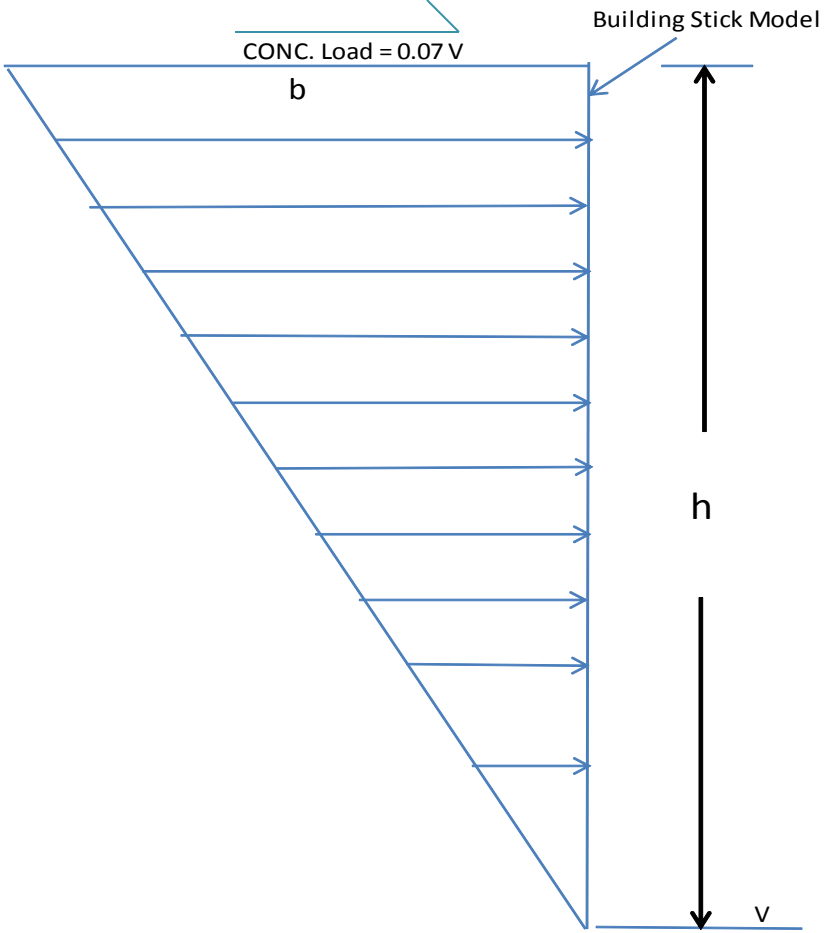
Initially commercial nuclear power reactor installations were governed by the building code of the country in which they were developed and installed. For example, the Connecticut Yankee NPP located at the low to moderate East Coast of the U.S., where detailed design began in 1963 was originally designed for a 0.03g earthquake acceleration of gravity and later was re-evaluated prior to operation for a 0.17g peak ground acceleration response spectrum.

1957 – 1965 Era

The Humboldt Bay NPP in northern California located in the high seismic region of the U.S. where detailed design started in 1959 was initially designed for a 0.25g static seismic acceleration and later re-evaluated for a 0.5g pga response spectrum earthquake. It is interesting to note that the original design of the Humboldt Bay NPP did not result in any seismic supports being required for piping systems that were required and installed when re-evaluated to the 0.5g acceleration in the 1970's.

1957 – 1965 Era

The initial seismic design loads during the 1957 to 1965 period were applied as static loads to define lateral or horizontal earthquake shear loads on structures as shown in Appendix A. The Appendix A load was defined as the base shear on building structures with this building lateral load distributed as an inverted triangle on the vertical projection of the building structure as shown in Figure 1. Seismic load design was generally limited to building structures.



Area of Triangular Load = $\sqrt{1/2(b \cdot h) = V(1 - 007)}$

where: h = height of the building structure
b = lateral seismic force per unit length of structure

FIGURE 1 **DISTRIBUTED STATIC SEISMIC LATERAL LOAD ON BUILDING**

1957 – 1965 Era

If distribution systems (piping, raceways, duct, etc.) or individual components were evaluated for earthquake loads, they were limited to applying the seismic acceleration to the mass distribution of the distribution system or component. The building code seismic design requirements summarized in Appendix A were meant to provide life safety to occupants of buildings and were not developed to assure continued operation or safety shutdown of high temperature and pressure industrial or power generating mechanical, electrical and instrumentation and control equipment.

1957 – 1965 Era

The period where building codes were used in NPP seismic design was relatively short lived. In 1963 in the U.S. a report TID-7024[1] was prepared for and published by the U.S. Atomic Energy Commission. This resulted in a very important change to the way seismic deign of NPP in that concepts of dynamic response spectra analysis were introduced to NPP design. Initially, these concepts were limited to building structures, but gradually were extended to the load definition for mechanical and electrical distribution systems and components in the 1965 – 1971 era.

1965 – 1971 Era

This era was ushered in by the publication of TID 7024, "Nuclear Reactor and Earthquakes," prepared for U.S. Atomic Energy Commission by Lockheed Aircraft Co. and Homes and Narver, 1963. This publication showed and explained the application of the Housner shaped ground response spectra as shown in Figure 2 to the dynamic analysis of building structures initially published by Housner in 1953. The Housner Spectra was based on an enveloping of the spectra developed from the time history acceleration recordings of 4 major earthquakes recorded in the western U.S. which were representative of soft rock and deep soil sites.

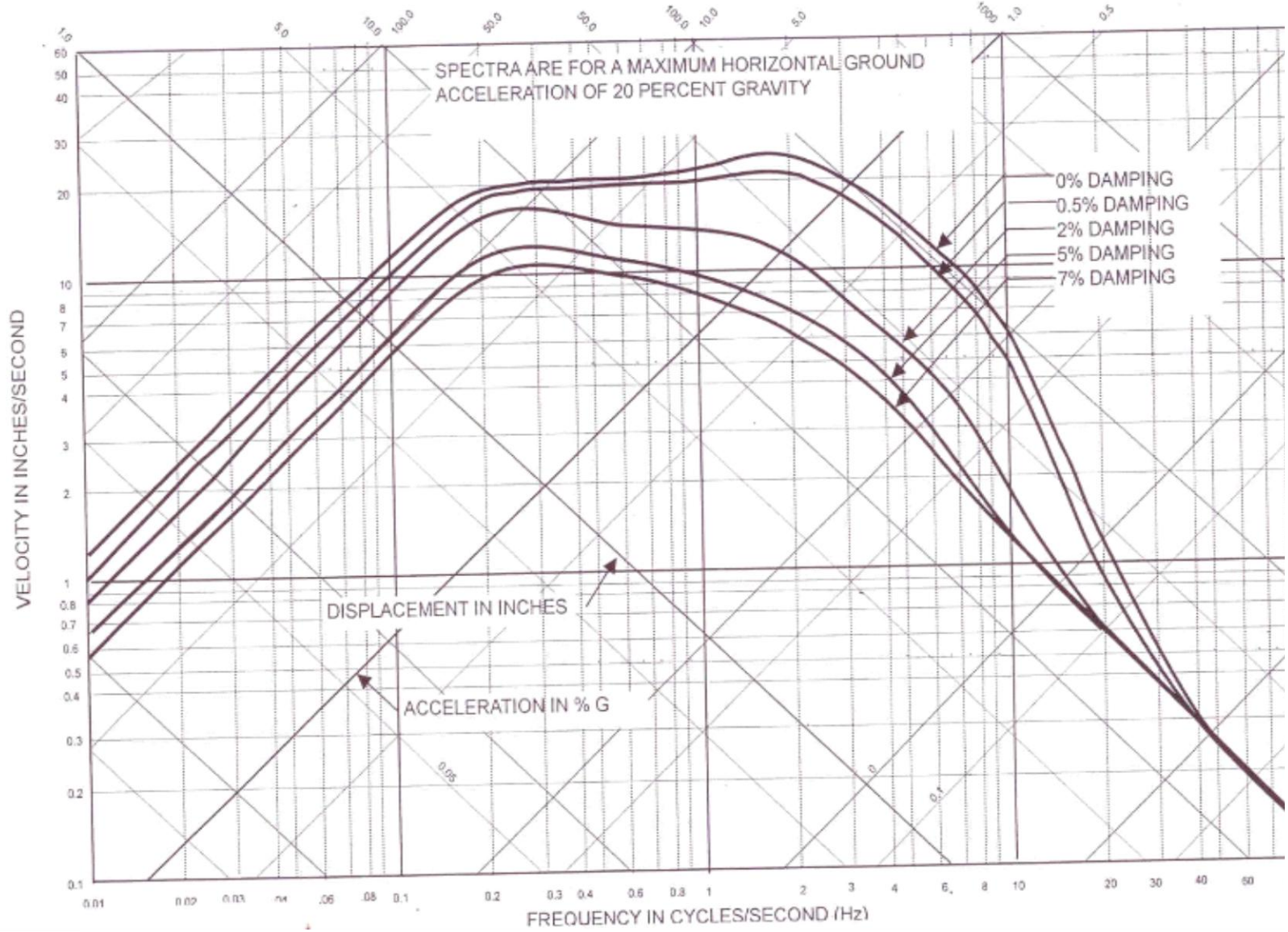


Figure 2 Housner Type Horizontal Ground Response Spectra Plotted on tripartite Paper Normal to 20% Gravity Peak Ground Acceleration

1965 – 1971 Era

To use the response spectra approach in design of the frequency response of the structures, distribution systems or components, SSC had to be known. The mathematic formulation to solve for natural frequencies of SSC were well known, but the calculation necessary to obtain these frequencies and individual frequency mode shapes, increased exponentially with the number of mass dynamic degrees of freedom considered.

1965 – 1971 Era

In 1962 the use of computers was at its infancy. To solve a 5 degree of freedom system using a $800.00 mechanical calculation capable of adding, subtracting, multiplying and dividing by hand took an engineer approximately 160 hours. As a result most seismic load calculations were made typically using the peak of the response spectra for the value of damping chosen to represent the damping characteristic of the SSC being evaluated.

1965 – 1971 Era

At the time, all analysis used the ground response spectra shape normalized to represent the peak floor acceleration at the support point of the SSC being evaluated. The term floor or in-structure response spectra with its shape modified by the response of the building to seismic ground input, was yet to be invented.

1971 – 2000 Era

The shape of the ground design basis response spectra gradually began to change with the evaluation of more actual earthquake data and the application of probability as the mean plus one standard deviation level used to define Design Basis Earthquakes as shown in Figures 3 through 5.

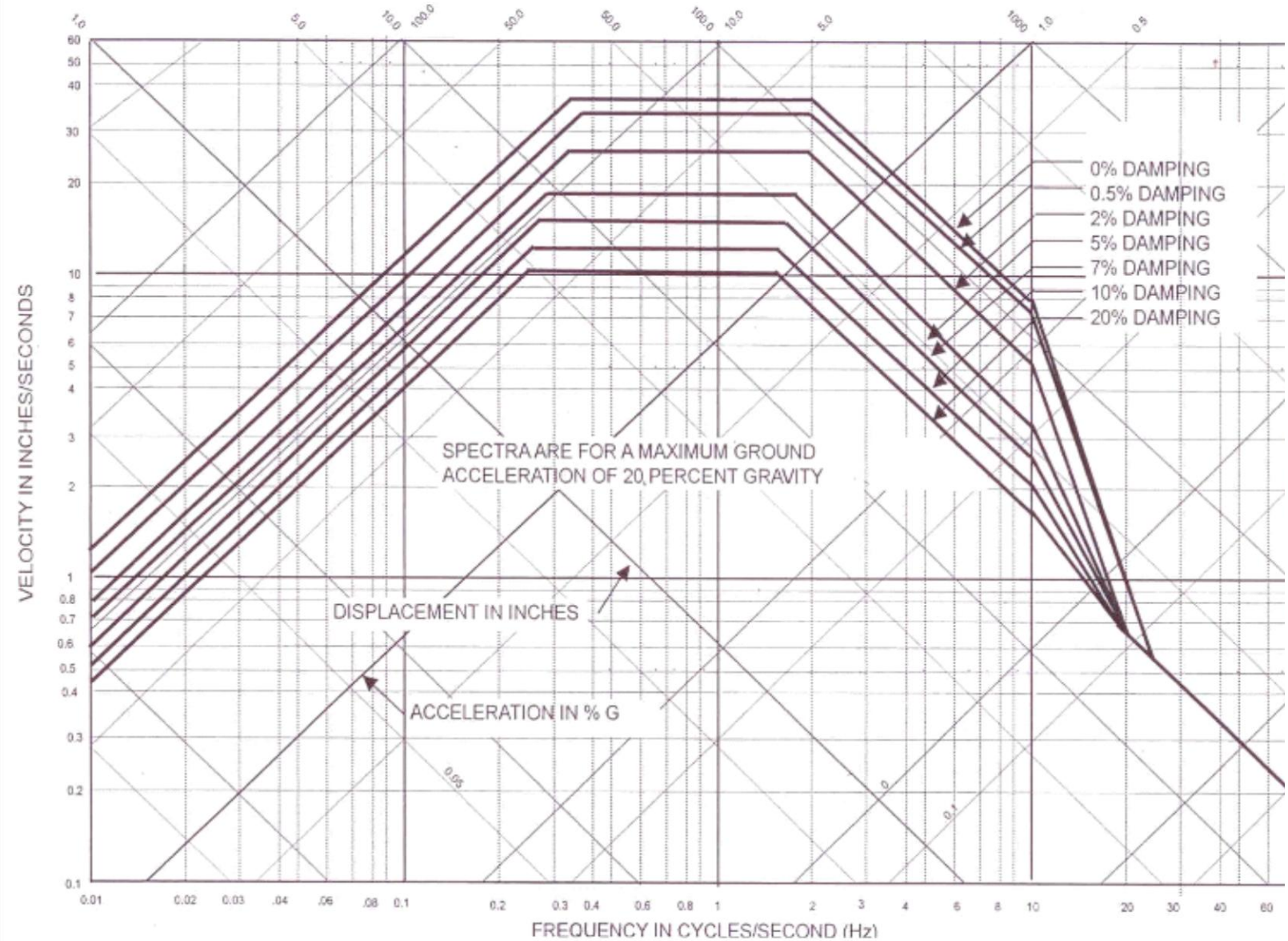


Figure 3 Original Newmark Response Spectra

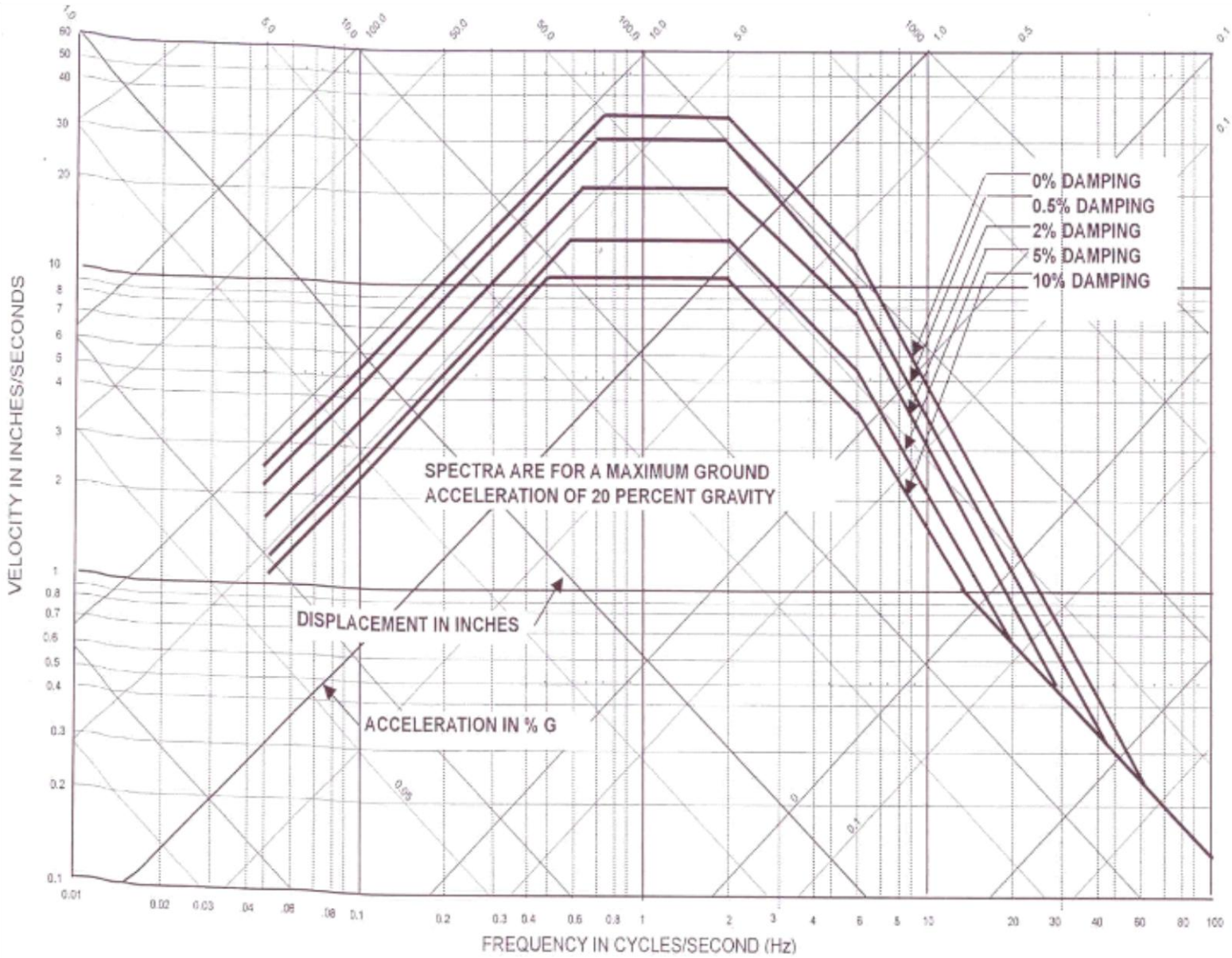


Figure 4 Later Newmark Response Spectra (1969)

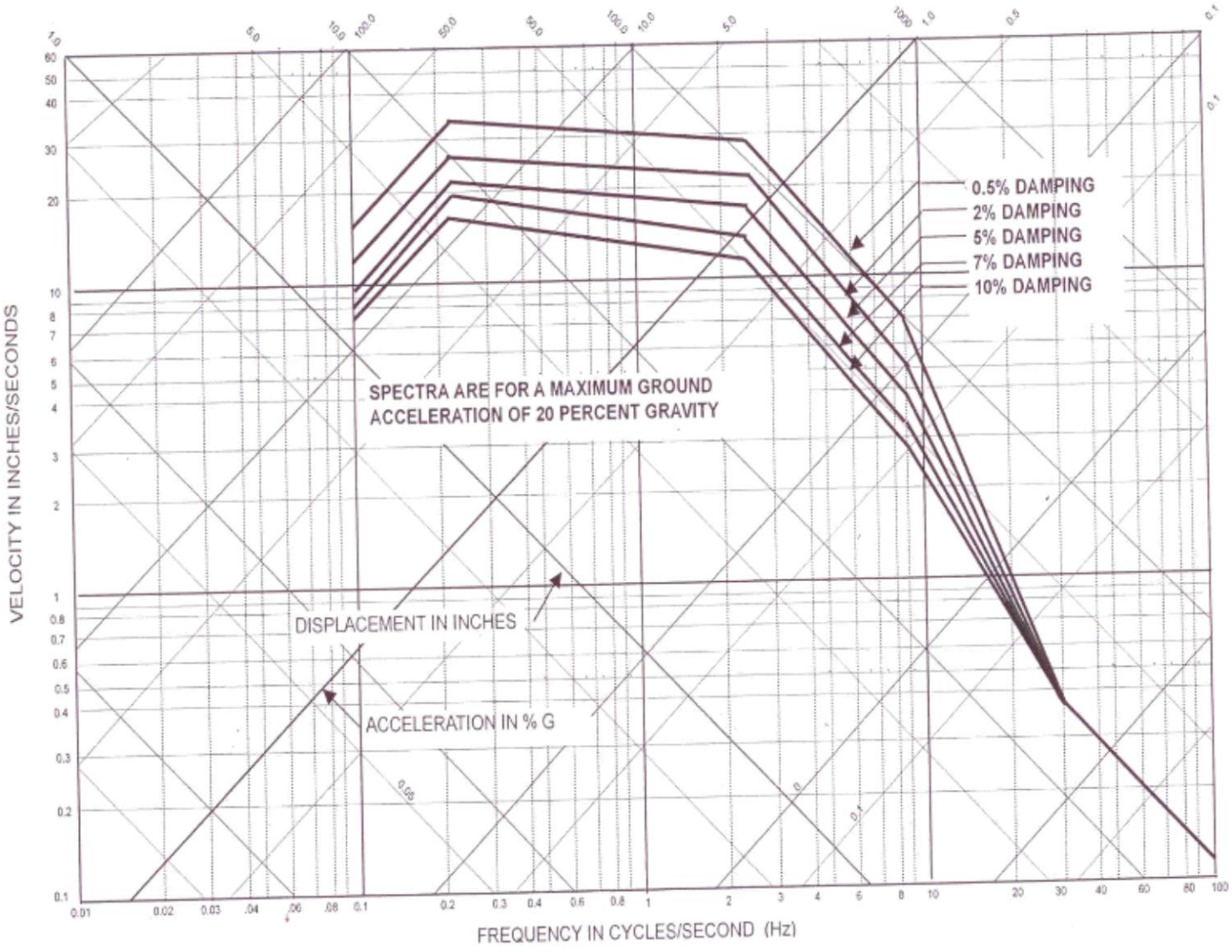
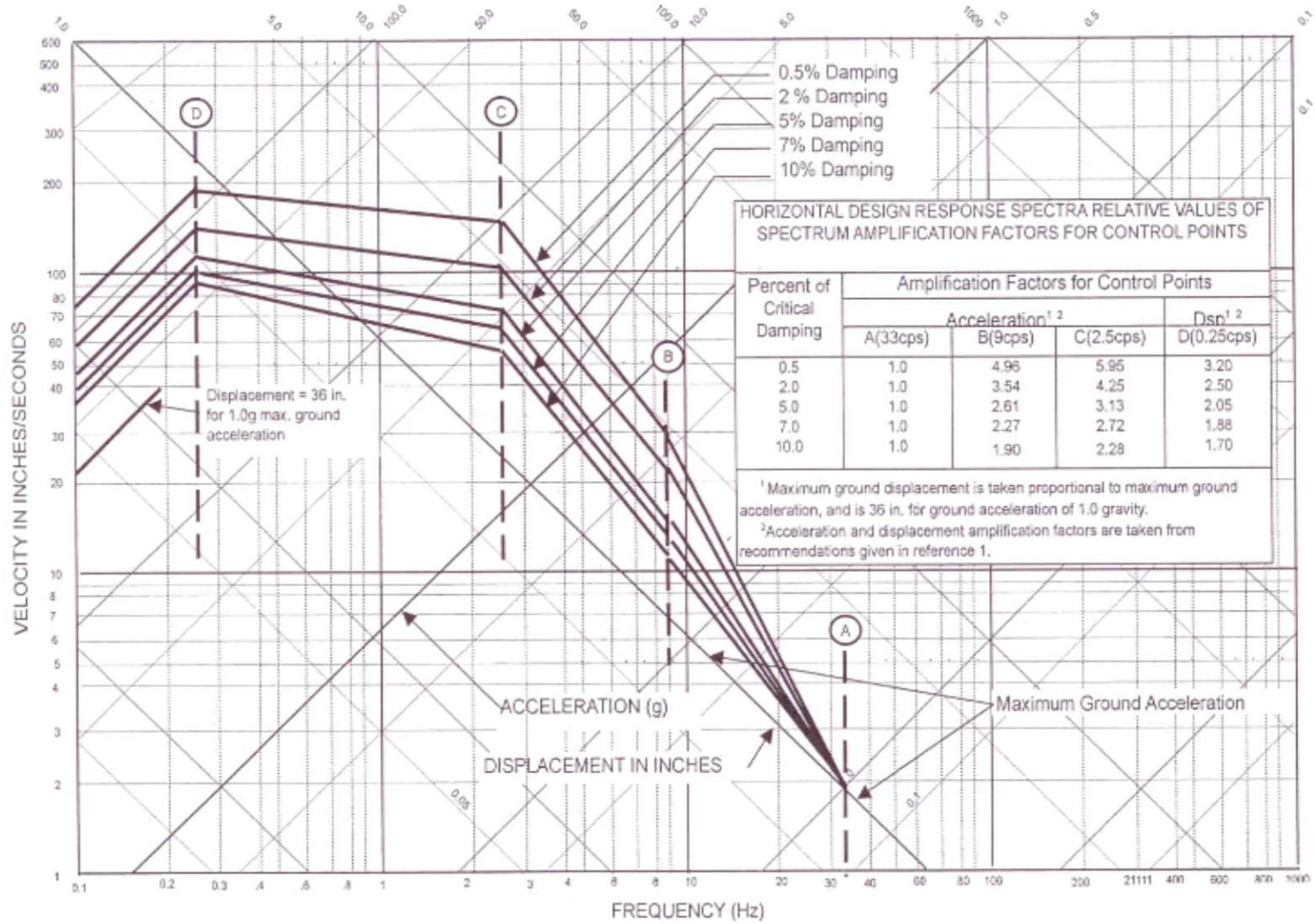


Figure 5 Newmark-Blume-Kappor Type Horizontal Ground Response Spectra Plotted on Tripartate Paper

1971 – 2000 Era

In 1973 a mean plus one standard design basis response spectra based on a Normal probability density function was established in the U.S. as shown in Figure 6. As a matter of convenience, the R.G. 1.60 spectral shape was also used to a considerable degree to design NPP outside the U.S. in situations where there was little or no measured earthquake motion experience or data available.



HORIZONTAL DESIGN RESPONSE SPECTRA RELATIVE VALUES OF SPECTRUM AMPLIFICATION FACTORS FOR CONTROL POINTS

Percent of Critical Damping	Amplification Factors for Control Points			
	Acceleration[1,2]			Dsp[1,2]
	A(33cps)	B(9cps)	C(2.5cps)	D(0.25cps)
0.5	1.0	4.96	5.95	3.20
2.0	1.0	3.54	4.25	2.50
5.0	1.0	2.61	3.13	2.05
7.0	1.0	2.27	2.72	1.88
10.0	1.0	1.90	2.28	1.70

[1] Maximum ground displacement is taken proportional to maximum ground acceleration, and is 36 in. for ground acceleration of 1.0 gravity.

[2] Acceleration and displacement amplification factors are taken from recommendations given in reference 1.

Figure 6 Horizontal Design Response Spectra Scaled to 1 G Horizontal Ground Acceleration from U.S. NRC R.G. 1.60

1971 – 2000 Era

Starting around 1970, floor or in-structure response spectra as typically shown in Figure 7 began to be used in design of SSC located at other than the free-field surface ground levels of buildings at other locations in the building which included the modify effect of foundation structure interaction for foundation media with shear wave velocities typically less than about 700 meters per second.

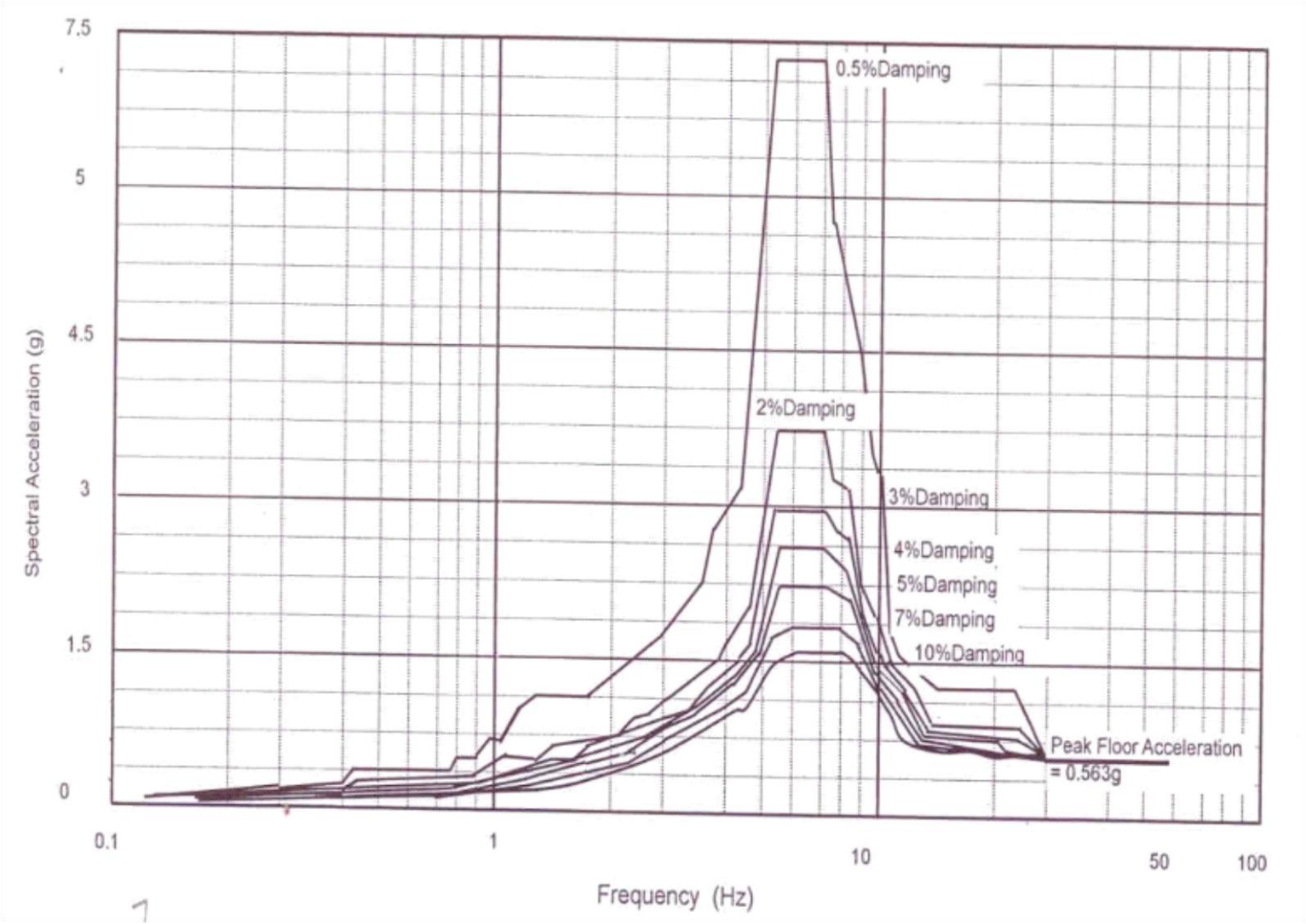


Figure 7 Typical In-Structure Mid-Height Acceleration Response Spectra for a Four-Story Building – 7% Building Damping and Equipment Damping as Shown

1971 – 2000 Era

Initially, the floor or in-structure response spectra were developed by applying a time history acceleration motion to the base of the building and recording the time history accelerations response at various elevations in the building structures.

1971 – 2000 Era

The validity of the input time history to the building structure in the free field was determined by using a time history motion (usually somewhat artificially modified) to generate the design basis ground response spectra which initially was a composite enveloping of several actual earthquake time histories (i.e. the R.G. 1.60 ground response spectra.

1971 – 2000 Era

Subsequently, procedures were developed where having the shape of the ground response spectra and the dynamic characteristics of the building structure, it was possible to directly generate a floor or in-structure response spectra without use of a time history input.

Era After 2000

This era is different from the eras preceding it in that there is an attempt to standardize the design of nuclear power plants to include not only a standardized reactor coolant system, but also the development of a standardized balance of plant. This resulted in the development of standardized ground response spectra and floor or in-structure response spectra seismic input to which the plant SSC are designed and constructed.

Era After 2000

These spectra are selected to be able to envelope a large number of potential specific site seismic response spectra such that the standardized plant could be constructed at a large number of different site locations and foundation conditions for geographical regions with moderate to low seismicity. The seismic design of nuclear safety related SSC at a particular NPP site in concept is reduced to the comparison of site specific spectra to the standard plant design spectra.

Era After 2000

It also began to be understood in this era that the shape of the design basis ground response spectra developed from actual recorded earthquake at relatively high seismic sites (i.e. California, Japan, Chile, etc.) were recorded at sites with a significant amount of faulting and other geographical and foundation characteristics which indicated the peak or dominate site response spectral acceleration typically occurred between 2.5 and 9 Hz.

Era After 2000

When actual earthquake motions are recorded at sites associated with moderate to low seismicity in the M_b = 5.0 to 6.5 range, the associated peak free-field ground acceleration response spectra peak at much higher frequencies; between 10 to 25 Hz. As a result, the standard plant free-field ground response spectra have tended to have higher acceleration spectrum amplitudes than existing standardized spectra (R.G. 1.60) used in the past particularly on rock sites.

SEISMIC ANALYTICAL MODELS

1957 – 1965 Era

In this era seismic lateral loads were defined statically as a function of the base shear on a building structure as shown in Appendix A. The V load was horizontally distributed as an inverted through on a stick model as shown in Figure 1 of the building having the same total stiffness as the building shear walls or frame running parallel to the direction of the lateral seismic loads in a nominal N-S and E-W direction. The resultant seismic forces on a particular shear wall or column line was distributed as a function of that wall or column line construction to the overall building lateral stiffness. In this manner the total lateral seismic load was applied to individual wall or column lines of the building.

1957 – 1965 Era

Vertical seismic loads were generally ignored during this period as well as explicit seismic loads on mechanical or electrical distribution systems or components other than to assure partial anchorage of the component. It should be understood that during this period the maximum acceleration even at a high seismic intensity site was limited to about 0.4g static applied acceleration at the roof of the structure and 0.2g at the center of gravity. The floor of structures were designed to have fundamental frequencies above 20Hz so that there was little or no amplification of vertical seismic acceleration (e.g. vertical seismic acceleration was taken as a constant with building height equal to 2/3 the horizontal pga).

1965 – 1967 Era

During the first half of this era mechanical and electrical components that were evaluated explicitly for seismic loads were modeled as one degree-of-freedom cantilever with the same shape in-structure response spectra as the ground response spectra except that the component damping was used and a pfa of the ground response spectral shape was normalized to equal the peak support point acceleration.

1965 – 1967 Era

In the later part of this era floor or in-structure response spectra began to be used which considered the filtering effect of the building response to the input ground response motion which resulted in a more amplified floor or in-structure spectrum at or near the dominate frequency of the building.

1965 – 1967 Era

It was also during this era that vertical seismic response came into general use. An effort was made to make all building structure floors to have a fundamental frequency above 20 Hz which resulted in little amplified seismic response in the vertical direction. During this period Japan continued to consider vertical seismic accelerations as a constant.

1965 – 1967 Era

The analytical models of the buildings in this era began to be dynamic in nature with more than one dynamic degree-of-freedom typically as shown in Figure 8 with each floor represented as a mass point and were initially assumed to have a fixed base, but soon began to consider foundation structure interaction where the shear wave velocity of the foundation media was less than 700 meter per second as shown in Figure 9.

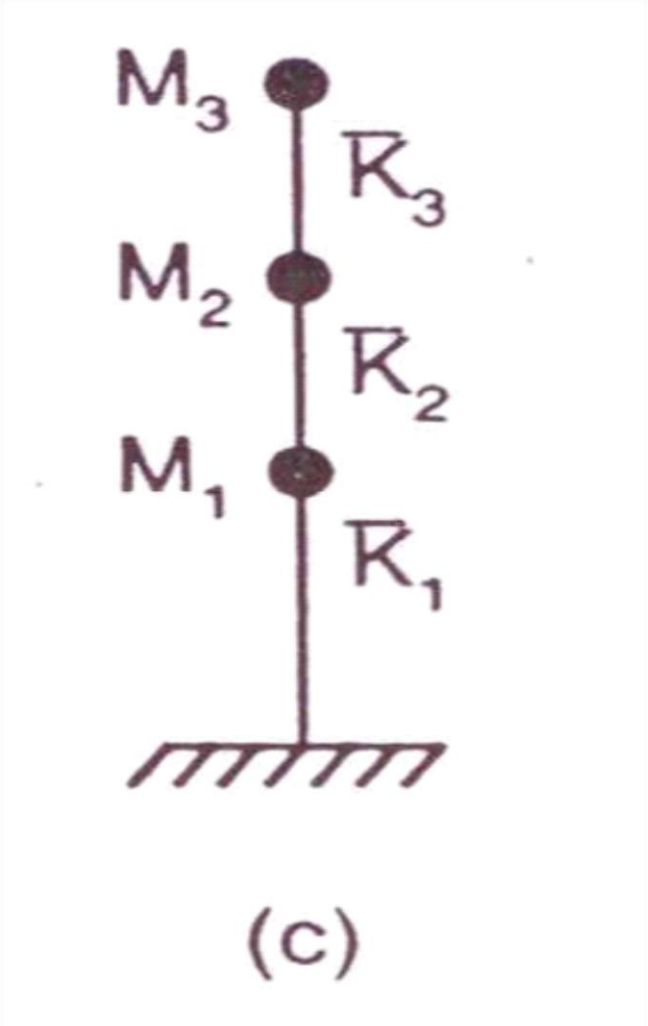


Figure 8 Beam or one-dimensional finite element model

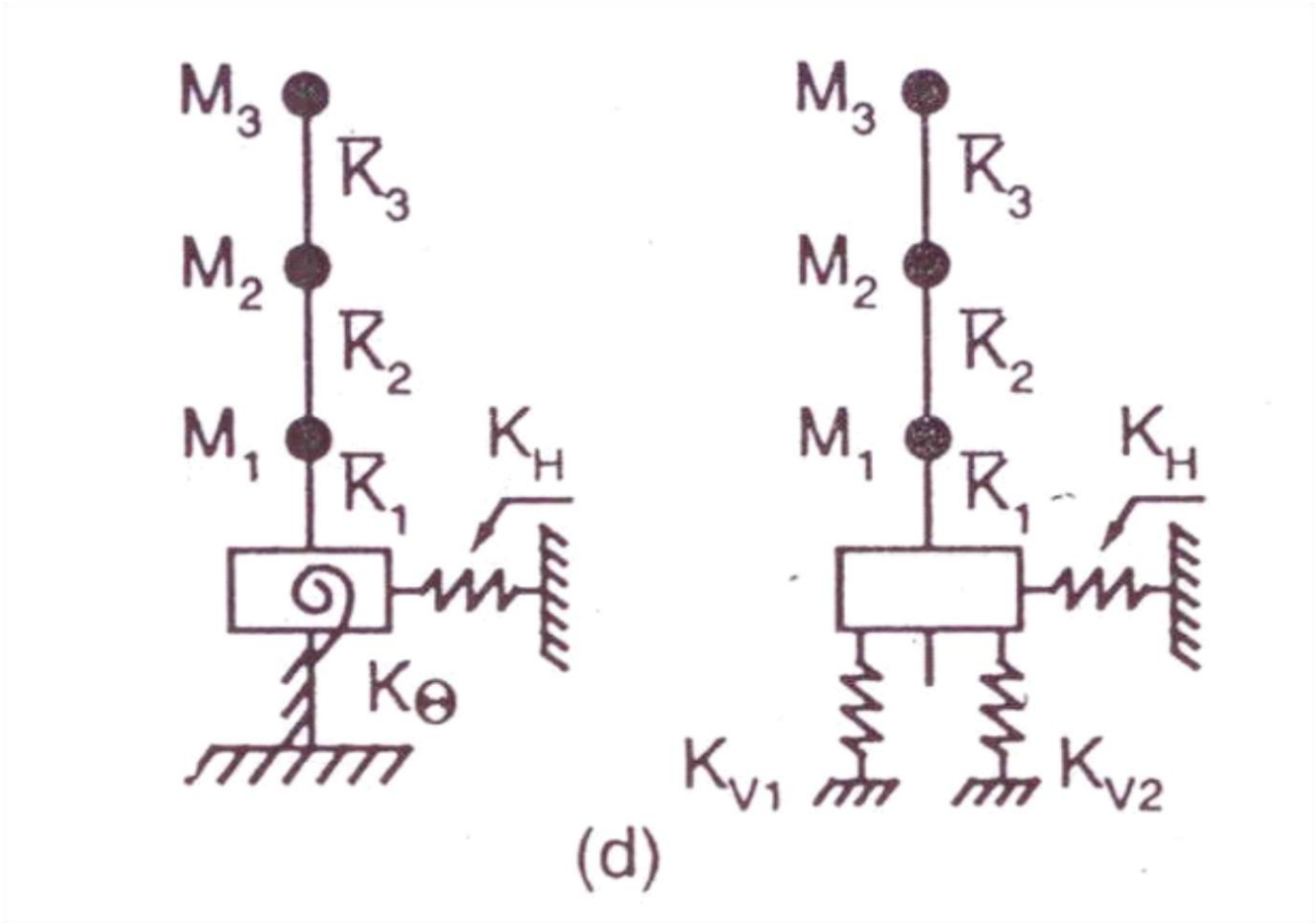


Figure 9 Beam or one-dimensional finite element on soil springs model

1971 – 2000 Era

In this period the building detailed static structural model transitional wind from one dominate finite element stick model in two space (one horizontal; one vertical) and two-dimensional finite element models in three-dimensional space. The dynamic models of the building tended to remain one finite element stick model, but now in a three-dimensional space as shown in Figure 10. soil structure interaction was typically represented by transfer function in the frequency rather than the time domain (SASSI).

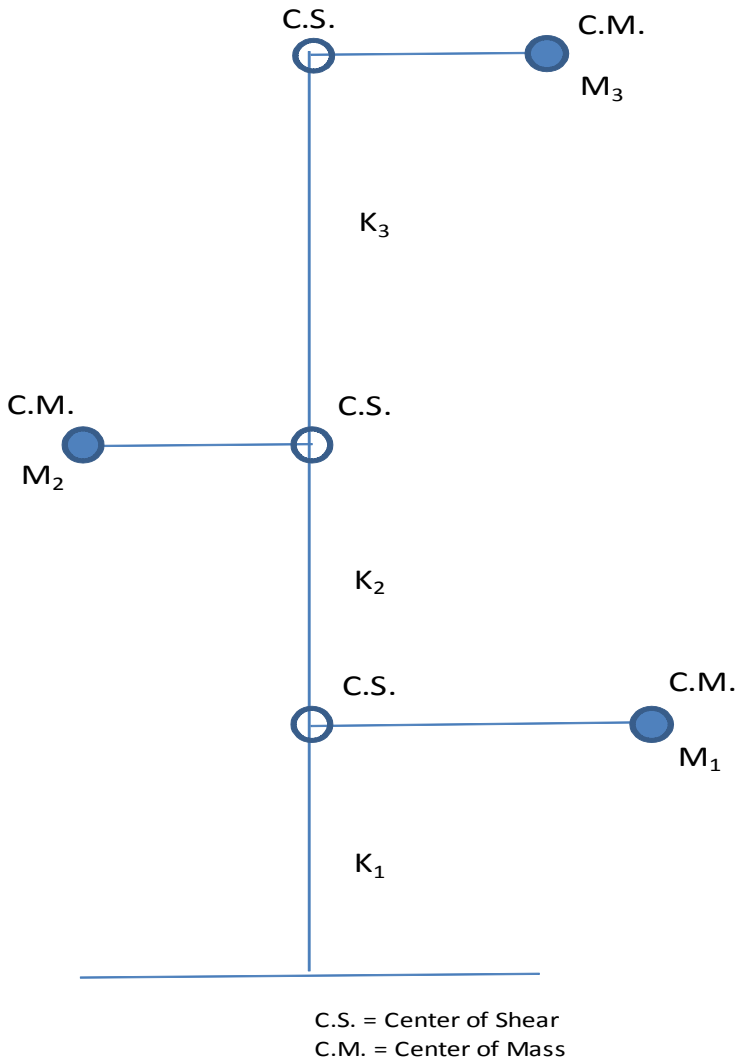


Figure 10 One-Dimensional Finite Element Including Response in 3 Directions

After 2000

Simple building structures were modeled dynamically as two-dimensional finite elements representing individual walls and slabs and one-dimensional beams and columns in three-dimensional spans which could be analyzed for seismic stress in induced members as shown in Figure 11. For complex building structures, the two step process is still used where seismic accelerations are determined based on simplified stick model representation of the building structure to determine resultant accelerations and then are applied to the detailed mass model to determine building member seismic moments or stresses, a more detailed finite element.

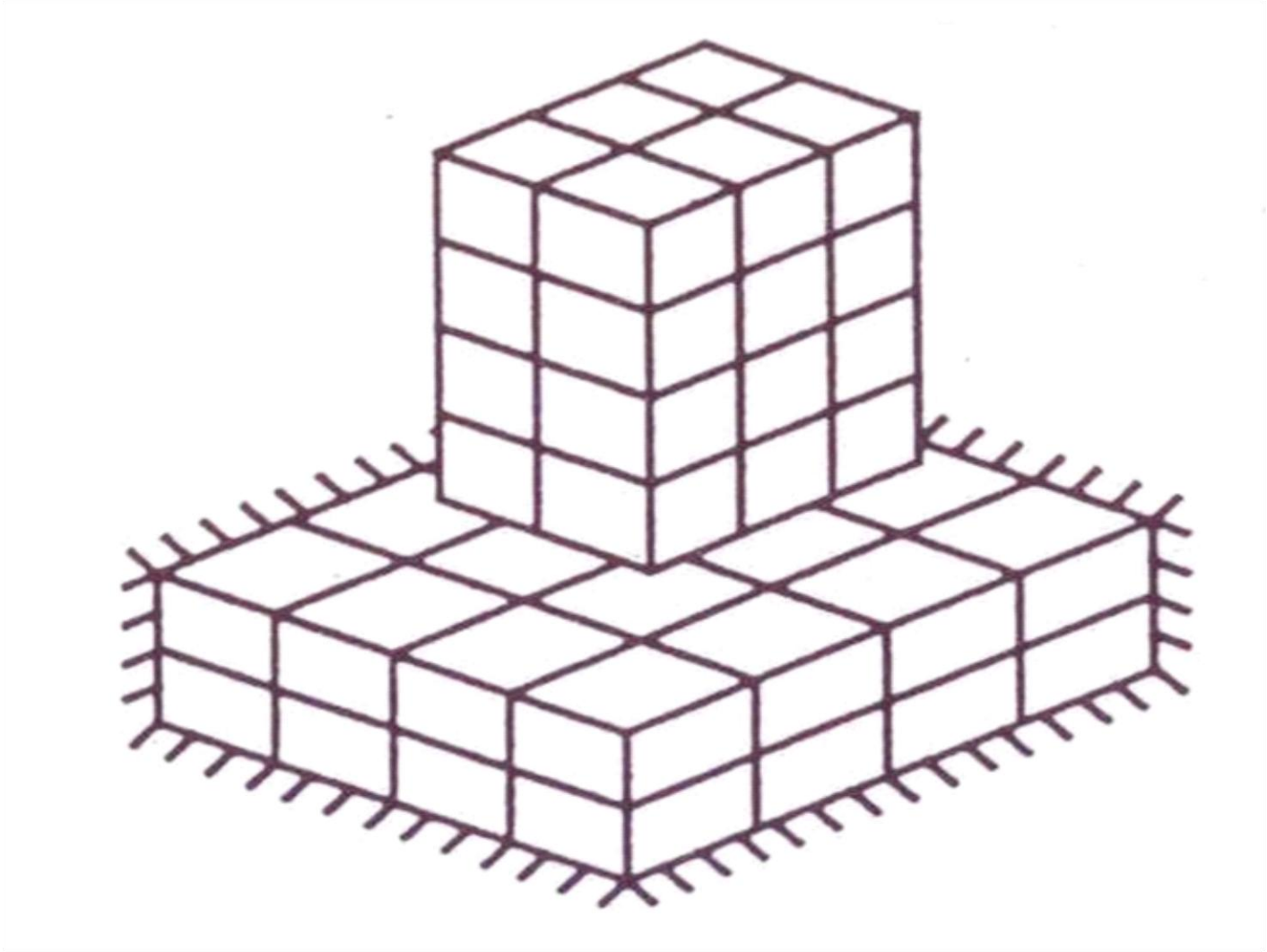


Figure 11

ACCEPTANCE CRITERIA

1957 – 1965 Era

During this era load combinations of dead plus live plus earthquake stress were considered in the same loading category and were permitted a 1/3 increase in normal allowable stresses. The normal allowable stresses in buildings were typically between 0.25 and 0.75 times yield or the equivalent of yield stress as a function of the national (reinforced concrete; structural steel) and the types of stress, tension, axial tension, axial compression bending.

1967 – 1971 Era

The Design Basis Earthquake load had been redefined as an extreme load category with the normal allowable stresses for buildings multiplied by 1.5 as acceptance criteria. For mechanical systems and components, resultant stresses were determined as principal stresses at 0.88 specified minimum yield stress for the combination of dead, live, pressure and earthquake loads. Temperature and other deformation limited stresses were limited to 2.0 times specified minimum yield stress.

1971 – Present Era

The acceptance criteria for buildings were effectively increased to 1.67 times normal allowable stresses and impactive and impulsive loads were allowed to go beyond yield as a function of provided ductility factors that range from 3 to 20.

The allowable stress for mechanical components and distribution systems for load combinations which consider dead plus live plus pressure plus DBE were based on principal stresses were taken as 1.6 yield stress for components and 2.0 times specified minimum yield stress for piping.

APPENDIX A

Summary of National Building Codes Which Consider Earthquake Resistant Design Of Nuclear Power Reactor States Circa Late 1960's

There are nine (9) factors which were potential candidates as important or influenced in the development of a base shear coefficient for seismic design of building structures for life safety protection. At the time there was essentially no consideration of the seismic design of mechanical or electrical systems or components. In formula form these factors were identified as follows:

$$V = [Z\ L\ I]\ ]K\ C\ R\ S\ F\ W]$$

where:

V = base shear coefficient

Z = seismicity geographical location coefficients typically associated with a 50 to 100 year return period

L = structural life factor

I = structural importance factor as a function of:

(a) cost

(b) occupancy

(c) hazard

K = ductility factor

- C = resonance effect with the earthquake dominate frequency
- R = mode participation factor
- S = soils or site amplification factor
- F = foundation design factor, and
- W = dead load plus some live load percentage typically taken as 25 percent

The national building code following discussion presents a summary of the basic parts of codes from various countries as extracted from the work of Mendenhall*. It mentions factors that are considered along with their values, as well as those that are not considered.

Canada	V = Z I K C S W – Six (6) Factors Considered			
	Zone	Z	Soil	S
	0	0	Not Highly Compressible	1.0
	1	0.025	Highly Compressible	1.5
	2	0.050		
	3	0.100		
	Importance		I	
	For buildings with large numbers of people, hospitals, power stations, fire stations, et cetera		1.3	
	All other buildings		1.0	
	Structure Type		K	
	Rigid frames which resist 50% of the shear, or walls which can carry shear ductility		1.75	
	All other types of buildings		1.25	
	$C = \frac{10}{9+N}$ ' $N = Total\ Number\ of\ Stories$			
	Note: Factors l, R and F are missing			

France	V = Z I C S F W – Six (6) Factors Considered

Z I are lumped and vary at the discretion of the designer between 0.033 – 0.26. These depend on the intensity required to protect the construction

$$C = 1/T^{1/3}$$

Soil	Foundation Type					
	A	B	C	D	E	F
Rocky	1.00	0.90	0.80	NA	NA	NA
Medium	1.15	1.15	1.00	0.90	1.00	110
G	1.25	1.25	1.15	1.00	1.15	1.30
H	NA	NA	NA	1.10	1.30	NA

where:

A = continuous spread footing

B = individual spread footing

C = drilled piles

D = raft

E = laterally supported piles

F = laterally unsupported piles

G = loose soil possessing sufficient strength to hold together in water

H = mud, silt, slime, saturated

Japan	V = Z K R S W – Five (5) Factors Considered

Z I are lumped and vary at the discretion of the designer between 0.033 – 0.26. These depend on the intensity required to protect the construction

Zone	Z	Building Height	R
1	0.12	< 52.8’	1.00
2	0.108	>52.8’	1.00 + 0.05N
3	0.906		

Where:

$$N = \frac{H - 52.8}{13.1}$$

$$H = Bldg.Ht.$$

Soil	K-Structure Type			
	Wood	Steel	A	Masonry
B	1.0	1.0	1.33	1.67
C	1.33	1.33	1.50	1.67
D	1.67	1.67	1.67	1.67
E	2.50	1.67	1.67	1.67

where:

A = reinforced concrete, steel frame and reinforced concrete or steel concrete composite

B = rock, hard sandy gravel, tertiary or older strata over considerable area

Rumania	V = Z I K C R S W – Seven (7) Factors Considered		
	Z I = 0.0225 – 0.090 depending on building importance and zone		
	R = complex factor depending on degrees-of-freedom		
	Building Type	K	
	Reinforced concrete frames and hinged roofs on reinforced concrete struts		
	All others	1.2	C = 1/T
	High chimneys, tanks, etc.	1.0	Min. = 0.667
		1.5	Max. = 3.33
	Soil		
	(Allowable bearing)	S	Limit on SC
	>4.096 KSF	1.0	No greater than 3.33
	<4.096 KSF	1.25	No greater than 3.33
	Silts water saturated soils above foundation level	1.50	No greater than 3.33
	Note: (1) two factors omitted (2) W includes total dead and live load (3) allowable bearing implies a foundation design factor		

Note: Rumania is included because it was the national building code with mostof the 9 factors considered

Russia	V = Z C R S W – Five (5) Factors Considered		
	Zone	Z	
	9	0.090	C = 1/T
	8	0.045	Min. = 0.667
	7	0.0225	Max. = 3.33
			R = complex function of weight and height
	Soil factors as such are omitted. Rather, the above values assume soil is medium; i.e. sandy, clayish with low ground water table. Detailed estimates of seismic intensities employ "micro-maps" on special instructions from populated points and industrial sites Most favorable soil – unweathered rocks, compact dry soil with large rock pieces. Unfavorable soil – saturated gravel, sand, clay with macropores, plastic or creeping clay.		
	Note: (1) L, I K and F are omitted (2) microzone maps are employed		

United States	V = Z K C W – Four (4) Factors Considered			
	Zone	Z	$C = 0.05/T^{1/3}$	
	0	0	Max. = 0.10 for 1-2 story structures	
	1	0.25		
	2	0.50		
	3	1.00	Building Type	K
			Shear wall	1.33
			Ductile space frame	0.67
			Combination of above	0.80
			all other	1.00
	Note: (1) L, I, R, S and F are omitted			

West Germany	V = Z S W – Three (3) Factors Considered		
	Soil Type	S	
		Zone 1	Zone 2
	Rock, gravel, coarse sand, and hard cohesive soil	0.05	0.05
	Medium and fine sand half-hard cohesive soil	0.075	0.05
	Stiff cohesive soil and pile foundations	0.10	0.05
	Note: (1) L, I, K, C, R and F are omitted (2) there is no soil effect in Zone 2		