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REVIEW OF THE DEVELOPMENT OF THE CAPACITY SPECTRUM METHOD

Sigmund A. Freeman
Wiss, Janney, Elstner Associates, Inc.
2200 Powell Street, Suite 925
Emeryville, CA 94608, U.S.A.

ABSTRACT

The Capacity Spectrum Method (CSM), a performance-based seismic analysis technique, can be used for a variety of purposes such as rapid evaluation of a large inventory of buildings, design verification for new construction of individual buildings, evaluation of an existing structure to identify damage states, and correlation of damage states of buildings to various amplitudes of ground motion. The procedure compares the capacity of the structure (in the form of a pushover curve) with the demands on the structure (in the form of response spectra). The graphical intersection of the two curves approximates the response of the structure. In order to account for non-linear inelastic behavior of the structural system, effective viscous damping values are applied to linear-elastic response spectra similar to inelastic response spectra. The paper summarizes the development of the CSM from the 1970s to the present and includes discussions on modifications presented by other researchers, as well as recommendations by the author.

KEYWORDS: Capacity Spectrum Method, Response Spectra, Damping, Ductility, Push-Over Curve

INTRODUCTION TO PBSD AND THE CSM

The purpose of Performance-Based Seismic Design (PBSD) is to give a realistic assessment of how a structure will perform when subjected to either particular or generalized earthquake ground motion. While the code design provides a pseudo-capacity to resist a prescribed lateral force, this force level is substantially less than that to which a building may be subjected during a postulated major earthquake. It is assumed that the structure will be able to withstand the major earthquake ground motion by components yielding into the inelastic range, absorbing energy, and acting in a ductile manner as well as by a multitude of other actions and effects not explicitly considered in code applications (Freeman, 1992). Although the code requires special ductile detailing, it does not provide a means to determine how the structure will actually perform under severe earthquake conditions. This is the role of PBSD (Freeman et al., 2004).

The Capacity Spectrum Method (CSM) is a procedure that can be applied to PBSD. The CSM was first introduced in the 1970s as a rapid evaluation procedure in a pilot project for assessing seismic vulnerability of buildings at the Puget Sound Naval Shipyard (Freeman et al., 1975). In the 1980s, it was used as a procedure to find a correlation between earthquake ground motion and building performance (ATC, 1982). The method was also developed into a design verification procedure for the Tri-services (Army, Navy, and Air Force) “Seismic Design Guidelines for Essential Buildings” manual (Freeman et al., 1984; Army, 1986). The procedure compares the capacity of the structure (in the form of a pushover curve) with the demands on the structure (in the form of a response spectrum). The graphical intersection of the two curves approximates the response of the structure. In order to account for non-linear inelastic behavior of the structural system, effective viscous damping values are applied to the linear-elastic response spectrum similar to an inelastic response spectrum. In the mid 1990s, the Tri-services manual was updated (WJE, 1996).

By converting the base shears and roof displacements from a non-linear pushover to equivalent spectral accelerations and displacements and superimposing an earthquake demand curve, the non-linear pushover becomes a capacity spectrum. The earthquake demand curve is represented by response spectra, plotted with different levels of “effective” or “surrogate” viscous damping (e.g. 5%, 10%, 15%, 20% and sometimes 30% to approximate the reduction in structural response due to the increasing levels of damage). By determining the point, where this capacity spectrum “breaks through” the earthquake demand, engineers can develop an estimate of the spectral acceleration, displacement, and damage that may occur for specific structure responding to a given earthquake. A number of changes have been proposed to the capacity spectrum method that increase the complexity and computational effort associated with this method, usually requiring iteration to find the “exact” point where the capacity spectrum intersects the “correct” level of damping. The author believes that iteration is unnecessarily complex and clumsy for the intended use of this procedure. Rather, the author views the capacity spectrum method as a tool for estimating and visualizing the likely behavior of the structure under a given earthquake in a simple graphical manner. By formatting the results in the acceleration-displacement-response-spectrum format (Mahaney et al., 1993) in lieu of the traditional spectral acceleration (Sa) versus period (T) format, the graphical and intuitive nature of the capacity spectrum method become even more apparent.

ORIGIN OF THE CSM

The CSM can trace its roots to John A Blume’s Reserve Energy Technique (RET) (Blume et al., 1961), which estimated the inelastic displacement by equating elastic energy (or work) with inelastic energy (or work) as illustrated in Figure 1. In other words, the area within the green trapezoid is equated to the area in the red triangle. The green line plateau is equal to the peak of the triangle divided by R. The ductility, μ (mu), is equal to the displacement at the end of the green line divided by the displacement at the bend in the green line. In the example shown in Figure 1, the elastic period is 0.70 sec and the inelastic secant period is 1.4 sec. The μ is equal to 4.0 and R is equal to 2.65. It should be noted that this procedure is consistent with the force/acceleration reduction factor $R = (2\mu - 1)^{½}$ associated with Newmark’s equation for the constant acceleration range of response spectra (Newmark and Hall, 1982).

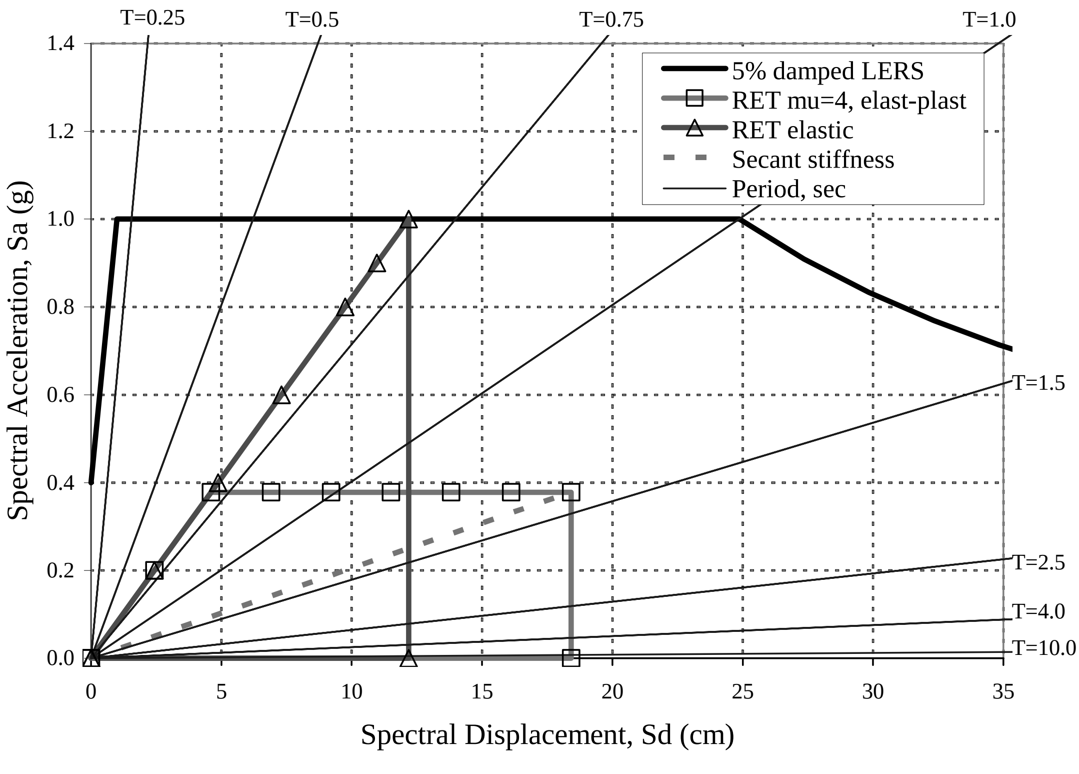

Fig. 1 John A. Blume’s reserve energy technique (RET)

1. Puget Sound Naval Shipyard (PSNSY) Project

In the early 1970s, while working for John A. Blume Associates, the author was assigned the task of evaluating 80 buildings for a pilot program on establishing the seismic vulnerability of the PSNSY (Freeman et al., 1975). The time allotted for each building was in the neighborhood of an average of six hours and the proposed procedure was to be the RET. Most of the buildings were built between the beginning of the 20th century and the early 1940s. There was a wide variety of building types including brick, concrete, steel, wood, moment frames, braced frames, shear walls, and combinations thereof.

Seismic design was most likely not considered for many of the buildings and in some cases there did not appear a definable lateral force resisting system. Many buildings had dual or multi-lateral force resisting systems, some acting in parallel and others acting sequentially.

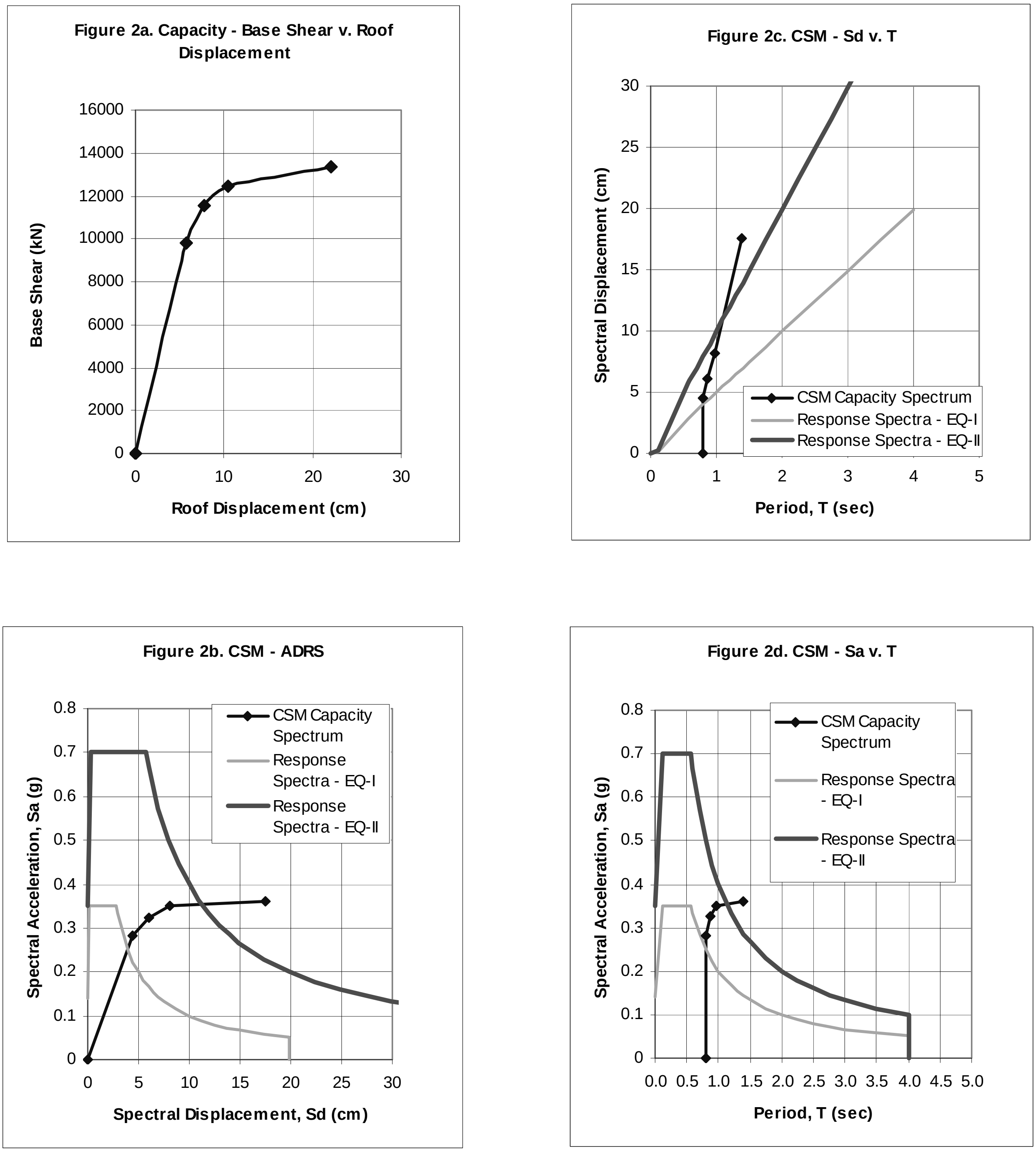

Fig. 2 Capacities and demand spectra

Capacities of the structures were determined on the basis of on-site observations, review of available drawings, calculations to approximate force-displacement relationships, and some engineering judgment. The capacity was defined by three points on a graph: origin, yield limit (incipient damage), and ultimate limit. This was essentially a rough approximation of what is now referred to as a pushover curve, which had been well illustrated in the 1961 PCA publication (Blume et al., 1961). For the PSNSY project, the yield limit was defined as the base shear represented by the force required to reach the capacity of the most rigid lateral force resisting element of the building systems. The ultimate limit was defined as the base shear causing the most flexible lateral force resisting elements to yield after the more rigid elements yielded or failed. It was possible for the ultimate base shear capacity to be lower than the yield limit capacity. However, the ultimate base shear capacity would represent a more flexible structure that would have a longer fundamental period and could be in a more favorable position in the response spectrum. A sample capacity curve is shown in Figure 2a, where the force is measured by base shear and the displacement is measured at the roof. For the PSNSY project, the capacity was represented by a bi-linear plot connecting the yield point and ultimate limit by a straight line.

Table 1: Return Periods and Probability of Exceedance

Probability of Exceedance for a Mean Return Period

Mean Return Period, Pr	**Probability of Exceedance in "x" Number of Years, P(ex)**				
	Number of Years, x				
(yrs)	**131**	50	**25**	10	1
2475	**5%**	2%	1.0%	0.4%	0.04%
475	24%	10%	5%	2%	0.2%
238	42%	19%	**10%**	4%	0.4%
87	78%	44%	**25%**	11%	1.1%
50	93%	63%	39%	18%	2%
25	99.5%	86%	63%	33%	4%
10	99.9998%	99%	92%	63%	10%
Equation: $P(ex) = 1 - \exp[-x/Pr]$					

Mean Return Period for a Probability of Exceedance

Probability of Exceedance, P(ex)	**Mean Return Period in "x" Number of Years, Pr**				
	Number of Years, x				
(% in "x" years)	1	10	**25**	50	**131**
1%	100	995	2488	4975	13035
2%	50.0	495	1238	2475	6485
5%	20.0	195	488	975	2554
10%	10.0	95.4	**238**	475	1244
25%	4.0	35.3	**87**	174	456
50%	2.0	14.9	36.6	72.6	189
85%	1.2	5.8	13.7	26.9	69.6
99%	1.0	2.7	5.9	11.4	28.9
Equation: $Pr = [1 - (1 - P(ex))^{1/x}]^{-1}$					

The demands on the structures were based on site specific probabilistic response spectra. The site had been subjected to ground motion from two moderate earthquakes from sources near Olympia, Washington in 1949 and 1965. Peak ground accelerations (PGA) at PSNSY were estimated to have been in the neighborhood of 0.07*g*. Results of a seismicity study concluded that (1) the maximum postulated PGA at the site is 0.20*g*, (2) the PGA for a 25-year period with a 10% probability of being exceeded is 0.07*g*, and (3) the PGA for a 25-year period with a 25% probability of being exceeded is 0.04*g*. The relationship between percent probability of exceedance in a number of years (e.g. 10%/25-yrs) can be translated into average return period (e.g. Pr = 238 years) by referring to Table 1. Similarly, for 25%/25-yrs, Pr = 87 years. Response spectra were given to represent these three PGAs. An estimated relationship between frequency of occurrence during a 131-year period and peak horizontal ground acceleration was also determined for the site. This was later used to compare average annual cost of earthquake damage to costs of seismic upgrades.

Because of the nature of the variety of building characteristics, it was determined that the RET methodology would not be applicable and that a different approach was needed. An intuitive approach, that seemed rational, was to graphically compare the demand to the capacity. By converting the capacity pushover curve from force-displacement to spectral acceleration (Sa)-spectral displacement (Sd), the capacity curve (i.e., a capacity spectrum) could be plotted on the same graph as the response spectrum (i.e., the demand). The conversion process to form the capacity spectrum is illustrated in Table 2 and the transformation from Figure 2a to Figure 2b will be discussed in more detail later.

Table 2: Capacity Spectrum

V Base Shear (kips)	d_R Roof Displ. (cm)	C_B V/W*	d_R/S_d Roof PF	C_B/Sa EMMR	S_a (*g*)	S_d (cm)	T Period (sec)
9786	5.79	0.22	1.30	0.78	0.282	4.45	0.80
11565	7.75	0.26	1.28	0.80	0.325	6.05	0.87
12455	10.41	0.28	1.28	0.80	0.350	8.14	0.97
13345	22.07	0.30	1.26	0.83	0.361	17.52	1.40

* Weight, W, equals 10,000 kips or 44,482 kN

PF is modal participation factor

EMMR is effective modal mass ratio

Table 3: Damping Values for Structural Systems

Structural System	Freeman et al. (1975)		Freeman et al. (1984)	
	Yield Limit	Ultimate Limit	Elastic-Linear	Post-Yield
Structural Steel	2%	5%	3%	7%
Reinforced Concrete	5%	10%	5%	10%
Masonry Shear Walls	2%	10%	7%	12%
Wood	5%	10%	10%	15%

For linear-elastic analysis, the pre-yield capacity is compared to the linear-elastic response spectra (LERS) viscously damped (e.g. 5% of critical damping). During excursions into the non-linear-inelastic range of response, energy will be absorbed by hysteretic damping. In addition, the non-linear behavior will prevent the resonant amplification assumed in the development of the LERS. Thus, there is a rational basis for reducing the response spectrum for inelastic response. How much of a reduction is a topic of debate that will be discussed later in this paper. For the PSNSY project, a very conservative approach was taken by using a modest increase in damping to represent the inelastic demand (e.g. increasing damping from 5% to 10% for a concrete structure, refer to Table 3, under the heading "Freeman et al. (1975)". The loss of stiffness that results from inelastic response will lengthen the effective period (T) that generally reduces the acceleration but increases the displacement. The process of reducing response spectra and taking into account period lengthening to represent inelastic response was based on observations and studies of buildings and recorded building response (Czarnicki et al., 1975; Freeman, 1978; Freeman et al., 1977).

The process of plotting the capacity spectrum with varying damped response spectra later became known as the Capacity Spectrum Method (CSM). The procedure is illustrated in Figure 2. Figure 2a shows a sample capacity curve. The curve represents the fundamental mode of vibration, which was appropriate for the buildings in the PSNSY study. The capacity curve is converted to a capacity spectrum (Figure 2b, Sa versus Sd) by means of dynamic modal participation factors as illustrated in Table 2. The capacity spectrum can also be expressed in terms of Sd versus T (Figure 2c) and Sa versus T (Figure 2d). The Sa versus T format was used for PSNSY because that was the common format for response spectra at that time. The response spectra in Figure 2 represent two earthquake demands. The smaller curve (lime green) represents a smaller earthquake (EQ-I) for which the building is expected to remain elastic (i.e., the response spectrum intersects the capacity spectrum below the yield point. The larger curve (red) represents the major earthquake (EQ-II), which has been adjusted by an appropriate damping factor to represent inelastic response (e.g. 10% damped instead of the 5% damped used for EQ-I). The fact that the capacity spectrum crosses the EQ-II spectrum satisfies the criteria (i.e., the capacity spectrum punches through the demand envelope to survive the earthquake demands). The point of crossing indicates the degree of estimated damage.

Sample values for reducing 5% damped spectra are shown in Table 4 and relationships between ductility, μ (mu), and damping, β (beta), are shown in Table 5 (adapted from Freeman (1998a)). A process for interpolating between damped response spectra and the intersection of the capacity spectrum will be discussed in detail later in this paper. More information on the PSNSY project is found in Freeman et al. (1975).

Table 4: Spectral Reduction Factors versus Damping (β)

Effective Damping (β), %	Newmark-Hall				*ECS-94 European*
	One-Sigma		Median		
	SRA	SRV	SRA	SRV	
5	1.00	1.00	1.00	1.00	*1.00*
7	0.86	0.90	0.91	0.93	*0.88*
10	0.72	0.80	0.78	0.83	*0.76*
15	0.58	0.70	0.64	0.73	*0.64*
20	0.46	0.60	0.55	0.66	*0.56*
25	0.38	0.53	0.48	0.60	*0.51*
30	0.32	0.47	0.42	0.55	*0.47*
35	.--	.--	0.38	0.52	*0.43*
40	.--	.--	0.33	0.50	*0.41*

Table 5: Effective Damping (β) versus Ductility (μ)

Effective Ductility, μ	Effective Percent of Critical Damping, β										
	WJE (1996)		ATC (1996) (r = 0 and 0.10)						From Figure 1 of Priestley et al. (1996)	New-mark Hall
	One-Sigma	Median	Type A		Type B		Type C			
			0	0.1	0	0.1	0	0.1		
1	5	5	5	5	5	5	5	5	5	5
1.25	7.5	8.5	18	16	13	12	9	9	--	8.3
1.5	10	12	24	23	18	17	11	11	--	11
2	14	16	33	29	25	22	16	14	13	17.5
3	21	26	39	33	29	25	19	16	17	27.5
4	26	35	40	34	29	25	20	16	19	35.5
6			40	33	29	25	20	16	21	46
8			40	31	29	24	20	15	22	54

2. Sample Buildings

After the PSNSY project was completed, the process was used for several case-studies where buildings had experienced ground motion that had been recorded. Included are two full scale four-story reinforced concrete test structures, built on the Nevada test site (NTS), that were subjected to ground motion from underground nuclear explosions (Freeman et al., 1976, 1977). Data available from the 1971 San Fernando earthquake was used for a CSM study of two reinforced concrete 7-story hotel buildings (Freeman, 1978). Variations of the CSM procedure were used in an Applied Technology Council (ATC) study on the correlation between earthquake ground motion and building performance (ATC, 1982). The CSM was applied to an example steel frame structure to compare varying design considerations to performance capabilities (Freeman, 1987). After the 1989 Loma Prieta earthquake, several buildings were evaluated using CSM (Mahaney et al., 1993). It was in this paper that Sa versus Sd format for response spectra (ADRS) was introduced as an alternative to the Sa versus T format.

3. Giving the Procedure a Name

Circa 1983, the process was being developed as a procedure for the seismic design of essential buildings as a supplement to the Tri-services (i.e., U.S. Army, Navy, and Air Force) Seismic Design manual. It was during this time that Dr. Peter Gergely had been reviewing the graphical concept described in the then available reference documents. In a telephone conversation, he expressed interest in the procedure, but suggested that to be successful it needed a name. He suggested “capacity spectrum method”, which this author accepted, and included it to describe the procedure in the upcoming manual discussed below. The late Professor Gergely had encouraged research at Cornell University on the validity of the CSM procedure. One example of such research is included in a thesis by G.R. Searer (Searer, 1994).

TRI-SERVICES DESIGN MANUALS

Over the years, the Departments of the Army, the Navy, and the Air Force have published technical manuals for design guidelines for new construction. The seismic design for buildings had generally followed the Uniform Building Code provisions as recommended by the Structural Engineers Association of California and modified by the Tri-services Committee. In the early 1980s, it was decided to provide guidelines for dynamic analysis provisions for essential buildings (e.g., hospitals, fire stations, etc.). The resulting “Seismic Design Guidelines for Essential Buildings” (Freeman et al., 1984; Army, 1986) provides a two-level dynamic analysis approach to design buildings. First, the building is designed to resist the lower level of earthquake motion by elastic behavior. Then the building is evaluated for its ability to resist the higher level earthquake with allowances for inelastic behavior. Two acceptable methods are presented, one being an updated and expanded version of the CSM approach initially developed for PSNSY. Recommended damping values were revised as shown on the right hand side of Table 3 (Freeman et al., 1984). The values of the primary, or more rigid, structural system governs; or if dual systems participate significantly, a weighted average can be used. Illustrative guidelines, such as a table similar to Table 2 and diagrams similar to Figure 2, are given. Detailed design examples are included and guidelines are given for nonstructural components as well as the structural system. The final document was published in 1986 (Army, 1986). It should be noted that the guidelines can also be applied to non-essential buildings as an alternative dynamic analysis procedure.

As building codes were being upgraded, it was decided circa 1993 to upgrade the 1986 document. At about midway into its development, the 1994 Northridge earthquake occurred. Advances were being made in the field of probabilistic and deterministic earthquake ground motion postulation and there was a greater interest in performance-based design procedures. In a quest to establish a rationale for surrogate damping values to represent inelastic response spectra, Nathan Newmark’s approach (Newmark and Hall, 1982) was reviewed. Using Newmark’s equations for determining damping reduction factors and those used for inelastic ductility reduction factors, an approximate relationship between percent damping (β) and ductility (μ) was established. Examples of these relationships are shown in Table 5, under the headings “Effective Ductility”, “WJE 1996”, and “Newmark-Hall”. For example, on the basis of a 5% damped LERS, an inelastic response spectrum for a ductility of 3 may be approximated by using roughly 27% damping. These values are substantially more liberal than those used for PSNSY and proposed in the earlier guidelines as shown in Table 3. Due to a delay in completing the updated guidelines and a change in Department of the Army policy of publishing technical manual, the final manuscript of the guidelines were not published. The final manuscript is presently being modified for possible distribution to interested parties (WJE, 1996).

ATC 40, SEISMIC EVALUATION AND RETROFIT OF CONCRETE BUILDINGS

Under the direction of the State of California, the Applied Technology Council (ATC) was assigned the task of developing guidelines for the seismic evaluation and retrofit of concrete buildings designed and constructed by earlier seismic design standards (ATC, 1996). As a result of a previous study, the CSM was selected as a recommended procedure. Initial drafts of the CSM procedure generally followed the format established in the Tri-services update (WJE, 1996). As the ATC document was being developed, the question arose regarding use of damped spectra representing inelastic spectra. There had been a school of thought that felt the surrogate damping values should be directly linked to hysteretic damping based on energy loss due to hysteretic cyclic behavior. When this method is used, the resulting inelastic reduction factors appear to most researchers to be too large (i.e., unconservative). In order to compensate for this concern, it was decided to identify three categories of reduction factors: Type A at 100%, Type B at two-thirds, and Type C at one-third the hysteretic reduction. It was generally accepted, at least by this author, that Types B and C would apply to existing concrete buildings and that the Type A 100% solution would not apply and could be resolved at a later date. The resulting proposed damping values are listed in Table 5 under the heading “ATC 1996”. Although the ATC document went through a detailed review process and was the subject of a number of workshops, its publication has created many ensuing interesting discussions and debates.

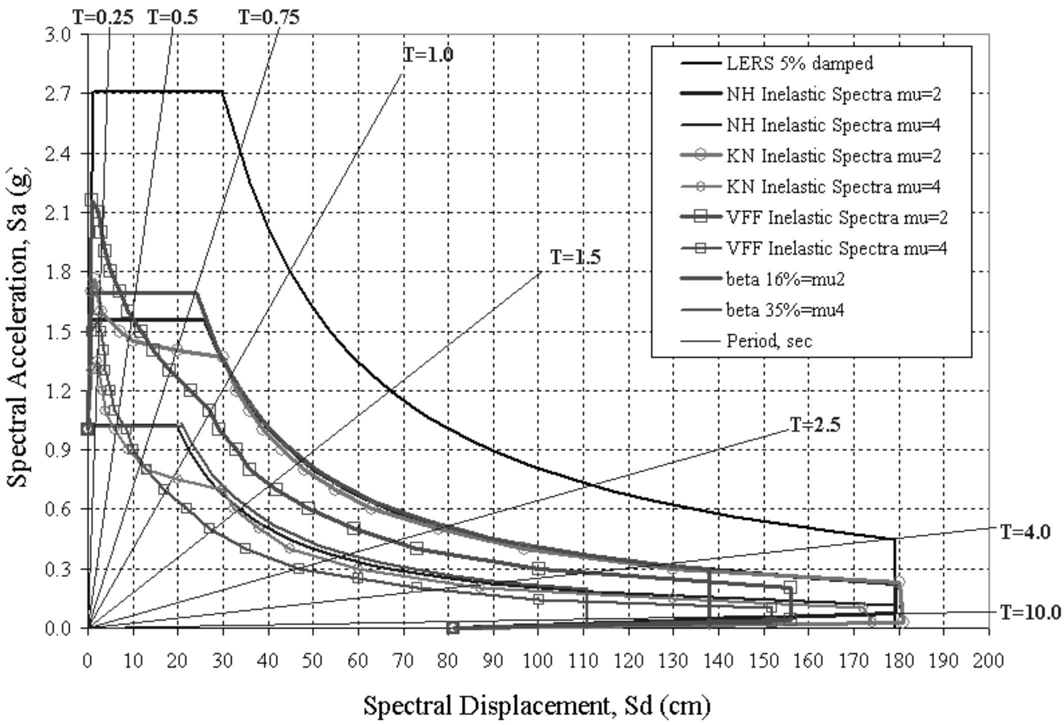

Fig. 3 Inelastic response spectra and damped spectra

CRITIQUES AND ALTERNATIVES

This author has watched with interest as groups of researchers discuss the fine points about which PBSD procedures give the “best” results. In recent years, there has been substantial research and discussion on the merits of inelastic response spectra and equivalent (surrogate) damped spectra and on the appropriateness of using damped spectra to represent inelastic response (e.g., Chopra and Goel, 1999; Fajfar, 1998; Judi et al., 2002). Although the conclusions of these researchers are not wholly consistent with each other, it has been claimed by some (Chopra and Goel, 1999) that use of damped spectra may lead to less conservative results as compared to inelastic spectra. The comparisons, in general, are based on the ATC 40 Type A damped spectra as discussed above. When comparisons are made with other surrogate damping procedures, such as those used in the PSNSY study and Tri-services manuals (Tables 3 and 5), claims of non-conservative results tend to disappear. An example is given in Figure 3. Two examples of inelastic response spectra (KN and VFF, from Chopra and Goel (1999)) are compared to Newmark-Hall ductility reductions and WJE 1996 equivalent damping, for ductility ratios of $\mu = 2$ and 4. As shown in Figure 3, the variations are not great (e.g. a 10% to 15% spread).

In many studies, researchers sometimes define an "exact" solution and compare other "approximate" solutions to this "exact" solution. Often, a non-linear dynamic time history is deemed to be the baseline "exact" solution. This author finds it difficult, however, to understand how any such solution is "exact." Every computer analysis requires development of a computer model of the building's structural system. In the computer, the model is then subjected to a digitized version of each of the ground motions. At best, the computer model is an idealized mathematical model that is based on a mélange of assumptions regarding the strength, behavior, and configuration of the component structural materials and assemblages. Therefore, the computer analysis is, at best, able to generate an exact numerical solution to a reasonable but inexact set of assumptions.

To anoint non-linear time histories as the benchmark against which all other non-linear methods should be compared is without basis. It is yet to be shown that any published non-linear time history evaluation accurately describes how a building has actually performed in a past earthquake, even though such an analysis is "after the fact" and benefits from prior knowledge of the building's actual performance. Recognizing that "after the fact" non-linear time histories are – more often than not – liberally tweaked in order to arrive at a reasonable facsimile of actual performance, it is dubious that a "before the fact" non-linear time history can be relied upon to correctly predict the response of a building to a yet-to-occur earthquake with unknown characteristics. To accurately predict how a building did or will perform, one must have accurate data on both demand and capacity; however, the accuracy of the answers cannot be greater than the accuracy of the input data. This is not to say that non-linear time history analysis does not provide important information about structural response; but it is not "exact".

It is important to try to understand why procedures, that appear to be rational, do not always give consistent results. In the case of inelastic response spectra, the peak displacements often include inelastic baseline shifts that can be very sensitive to the hysteretic model used in the analysis (Freeman et al., 2004). Interesting research on this topic, as well the use of equivalent damping, has been done by Hayder Judi and his associates (Judi et al., 2002 and correspondence by email).

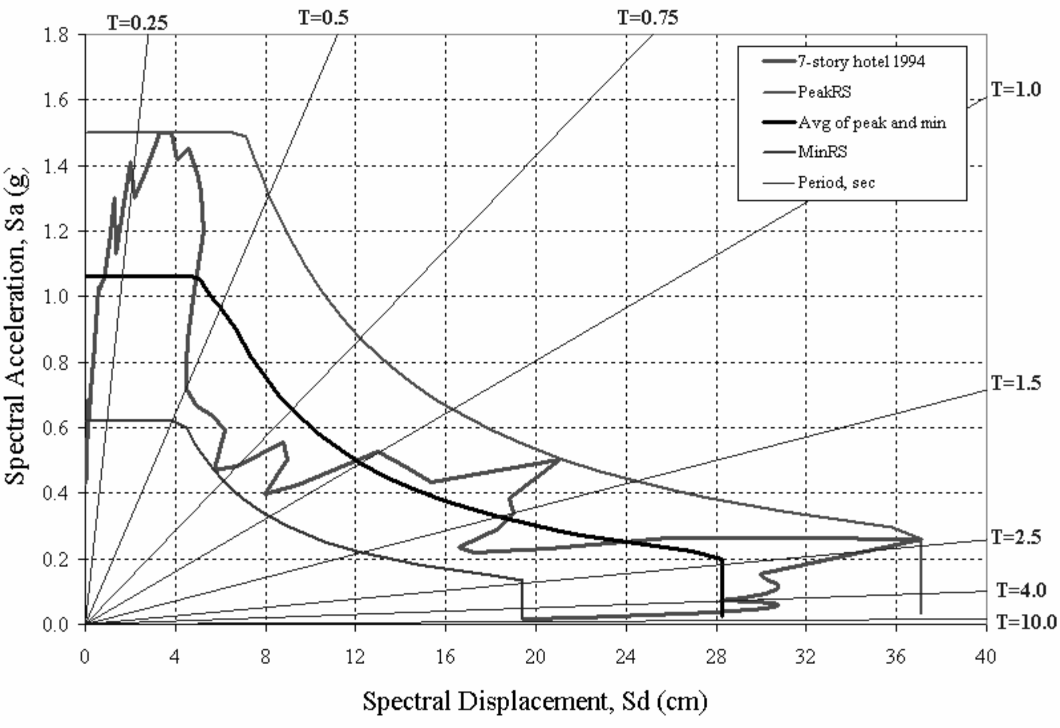

Fig. 4 7-story hotel: smoothing spectrum - Northridge 1994

As PBSD procedures become common, engineers must also be aware of the approximate nature of using smooth design response spectra as a basis for design of a structure to perform in a future earthquake. An example is given in Figure 4, where the green jagged 5% damped response spectrum represents the ground motion experienced in the 1994 Northridge earthquake. The upper (red) curve represents a code type smooth spectrum that envelopes the spikes of the measured spectrum, while the lower (purple) curve represents a lower limit defined by the valleys. The average between the maximum and minimum envelopes is represented by the heavy (black) curve. It can be seen that there can be vast differences between the potential response and a code projected response, depending on the period of the structure. The approximate nature of any PBSD procedure must be taken into account when designing buildings.

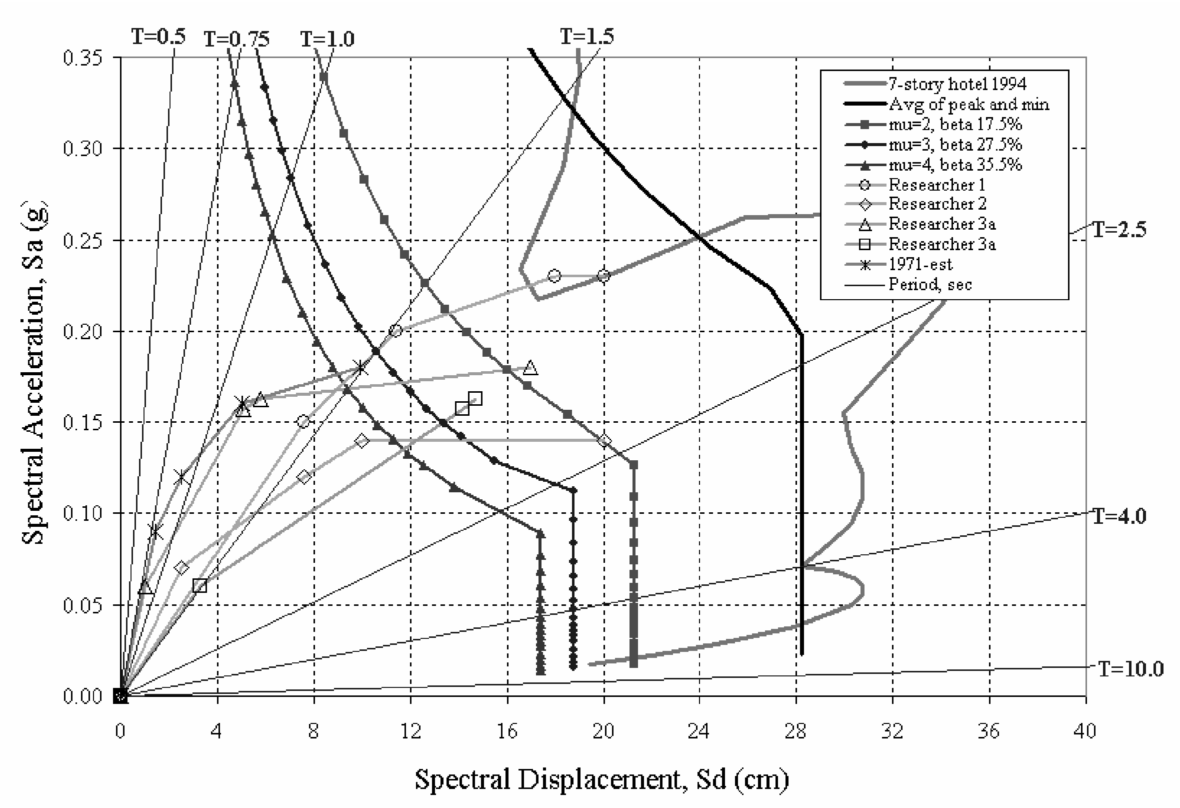

Fig. 5 7-story hotel: CSM study - Northridge 1994

An example of how calculated pushover curves can vary is illustrated in Figure 5. The graph is a close up of Figure 4, where the irregular (green) curve represents the earthquake spectrum and the heavy smooth (black) curve represents the average smooth spectrum. As a part of the CSM procedure, damped spectra at β = 17.5%, 27.5% and 35.5% are shown to represent ductility ratios of μ = 2, 3 and 4, respectively. Also shown are representations of capacity spectra developed by several researchers for the subject 7-story building that was damaged by the 1994 Northridge earthquake (Gilmartin et al., 1998). Although there appears to be substantial differences in the calculated capacity spectra, there is general agreement that the building was subjected to damaging ground motion and would have to exhibit ductility ratios of 3 or 4 to survive. Figure 6 illustrates a graphical procedure for estimating inelastic displacements by CSM (Freeman, 2000). By matching ductility ratio markings on the capacity spectrum with the closest effective damped spectrum, the ductility demand of 2.5 and the displacement of 16 cm can be estimated.

Alternatives to CSM include Priestley's direct displacement-based design method (DDBD) and Aschheim's yield point spectra (YPS). In a sense, DDBD (Priestley et al., 1996) is a reverse process of CSM. The target displacement and ductility ratio is selected to determine the required elastic period and strength required for the design. Equivalent viscous damping is used as shown in Table 5. They appear to be most consistent with ATC 40 Type C. The YPS procedure (Aschheim, 1999) is an extension of CSM, but it makes use of standardized inelastic spectra. In a sense, it is a cross between CSM and DDBD. The inelastic spectral displacements are reduced by the ductility ratios to form an elastic design spectrum.

PBSD procedures are generally limited to fundamental modes of vibration. For tall buildings, the participation of higher modes can be significant. Procedures for including higher mode effects have been presented that are based on the CSM concept (Paret et al., 1996; Sasaki et al., 1998). Other researchers are addressing this important issue.

Another resource for developing and verifying PBSD procedures is the use of building response records (Gilmartin et al., 1998). By carefully studying recorded building motion records, modal responses can be filtered out and pushover characteristics can be identified.

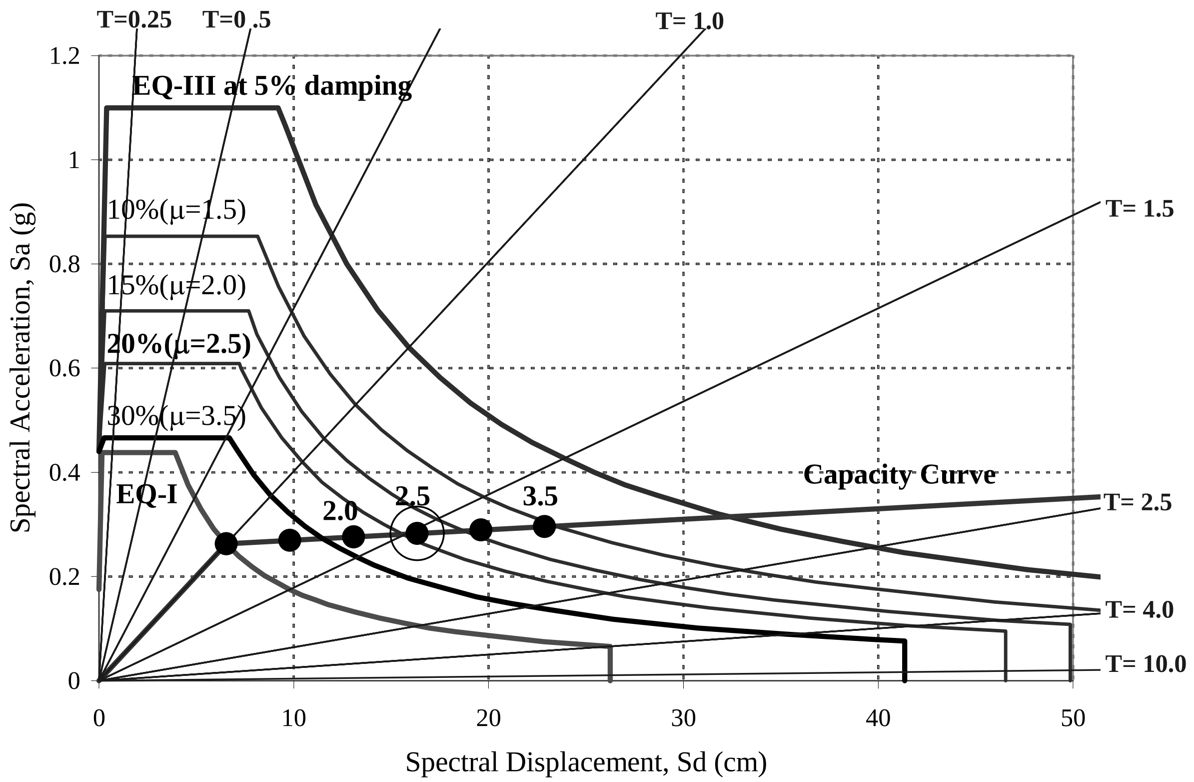

Fig. 6 Capacity spectrum method (CSM)

CONCLUSIONS

Over the years, PBSD procedures have evolved significantly from their humble beginnings; however, there has been a push to develop increasingly complex, codified PBSE procedures. In the author's opinion, by codifying PBSD, the very essence of PBSD, i.e. the focus on the attributes and behavior of an individual building, is destroyed. Engineers must be given sufficient latitude to arrive at the best estimate of a building's capacity (Freeman et al., 2004).

PBSD can be a useful tool for design and to estimate the performance characteristics of buildings subjected to strong earthquake ground motion. There is no "magic bullet" single procedure. It takes a combination of analytical procedures, data evaluation, judgment, experience and peer review to get a credible approximation of how a building works in the inelastic range of lateral motion (Freeman and Paret, 2000).

The CSM stands up well when compared to other PBSD procedures and has the added advantage of giving the engineer the opportunity to visualize the relationship between demand and capacity. Differences between the various methodologies have more to do with unknowns in material behavior and quantification of energy dissipation than in the methods of analysis (Freeman, 1998b).

Research on PBSD procedures should be encouraged to close the gap between researchers and practicing structural engineers. There should be more interaction between the researchers and practicing structural engineers to resolve controversial issues and to form consensus.

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