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THE FOLLY OF THE 0.85V MINIMUM IN THE DYNAMIC ANALYSIS PROVISIONS- CASE IN POINT: TALL CONCRETE CHIMNEYS

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ABSTRACT

In the current ASCE 7 building code the earthquake provisions require the base shear resulting from dynamic analysis to have a minimum value of 85% of the base shear determined from the equivalent lateral force procedure (ELFP). If one follows the development of the earthquake code provisions it is obvious that ELFP, as the name applies, is an attempt to be equivalent to a dynamic analysis procedure such as modal analysis. Thus, if the modal analysis is done correctly, with a proper design response spectrum, the results should be acceptable and not required to satisfy a minimum based on the ELFP. However, over the years of code development, code writers feared that engineers would not fully understand or would misuse the dynamic analysis provisions. Thus, minimum requirements were established for periods and base shears. Some of these concerns were justified, especially in the early years of dynamic analysis usage. However, there are certain cases where buildings or other structures do not fit the general dynamic characteristics, of a typical building. A case in point is those of tall reinforced concrete chimneys covered by ACI Committee 307. The paper examines the performance of several sample chimneys using ACI 307-8 seismic provisions for sites in regions of high seismicity. Although most of the sample chimneys satisfied the ASCE 7 minimum base shear requirement without adjustments, for several cases the minimum 85% of ELFP base shear V provision required significant increases in lateral forces. The results of this study concluded that the 0.85V rule is not a valid limitation for the modal analysis procedures for chimneys, as well as other structures. It is recommended that the limitation issues should be revisited and revised to be consistent with the modal analysis procedures.

Introduction

Ground motion and seismicity criteria have developed over the past 40 years from simple seismic zonation to detailed response spectra parameters. In order to make a rational comparison, one must review the development of seismic design provisions to understand how they progressed to their current state. Current seismic code provisions for the United States had their

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beginnings in the 1950s with the recommendations of the Structural Engineers Association of California (SEAOC) and their adoption into the Uniform Building Code (UBC). This work was supplemented with government sponsored development in the 1970s by the Applied Technology Council (ATC) and FEMA (NEHRP).

In the 1950s, the seismic design provisions were primarily an appendix to the building code. There was no direct relationship to ground motion, per se. The lateral force coefficient on a single-story building in the highest seismic zone was 0.133. The equivalent total lateral force coefficient was smaller for multi-story buildings ($V=CW$, $C=0.60/(N+4.5)$).

In the 1960s, major changes were made to the UBC (ICBO, 1961) earthquake provisions on the basis of recommendations of SEAOC (SEAOC, 1958) ($V=KCW$, $C= 0.05/T^{1/3}$).

In 1976, greatly influenced by the experience of the 1971 San Fernando earthquake in southern California, the UBC earthquake provisions were again subjected to major revisions. A soil factor S was added to the lateral force coefficient to account for the effects of the soil profile on the seismicity of the site. S was calculated by an equation that was dependent on the ratio of the characteristic site period and the period of vibration of the building ($V=ZIKCSW$, $C= 1/(15*T^{1/2})$).

The 1988 edition of the UBC also included major changes to the seismic provisions. The Z factor is presented as a measure of the PGA at “rock” sites: $V=ZICW/R_w$, $C=1.25*S/(T^{2/3})$. A lower limit on C/R_w was set 0.075. This results in a 3% base shear coefficient in California for Z=0.4g that is somewhat of a throwback to the California Riley Act (SEAOC 1967).

During the period of 1988-1997, modifications have been made to the soil-site factors and near-field effects in zones of highest seismicity have been added. In 1997 the UBC went from “allowable stress” design (R_w) to “strength” design (R) ($V=CvIW/RT$).

On the basis of the ATC 3-06 document developed in the 1970s (ATC 1978), FEMA (NEHRP 1997-2003) established guidelines that eventually lead to IBC 2000-2006 seismic procedures. The basis for the design response spectra becomes two-thirds of the MCE instead of the 475-year average return period generally accepted as being the basis for all previous editions of the UBC and the NEHRP. The MCE is defined as the lesser of the 2475-year average return period or 1.5 times the mean deterministic earthquake. Seismic response parameters S_s and S_1 are taken from contour maps or a database. These changes can have a substantial influence on defining the seismic hazards.

Important to note is that the shape of the “C” curve has changed over the years from the inverse of period T to the power of 1/3, 1/2, 2/3, to 1.0. These differences are illustrated in Fig. 1 and discussed later. It is also noted that provisions for dynamic analysis were include in the codes in the 1980s (SEAOC 1988) that required the use of response spectra.

As stated above, the basis for the development of current seismic building code provisions had their beginnings in the 1950s (Freeman 2007). A Joint Committee of the San Francisco Section of ASCE and the Structural Engineers Association of Northern California prepared a “model lateral force provision” based on a dynamic analysis approach and response spectra

(Anderson et al, 1952). The Proposed Design Curve, $C = K/T$, was based on a compromise between a Standard Acceleration Spectrum by M. A. Biot (Biot 1941, 1942) and an El Centro Analysis by E. C. Robison (Fig. 2). It is interesting to note that the Biot curve PGA of 0.2g has a peak spectral acceleration of 1.0g at a period of 0.2 seconds. The curve then descends in proportion to 1/T (i.e., constant velocity). If the peak spectral acceleration is limited to 2.5 times the PGA, the Biot spectrum is very close to the 1997 UBC design spectrum for a PGA of 0.2g. A design lateral force coefficient was C=0.015/T with a maximum of 0.06 and minimum of 0.02 was proposed. These values were considered consistent with the then current practice and the weight of the building included a percentage of live load.

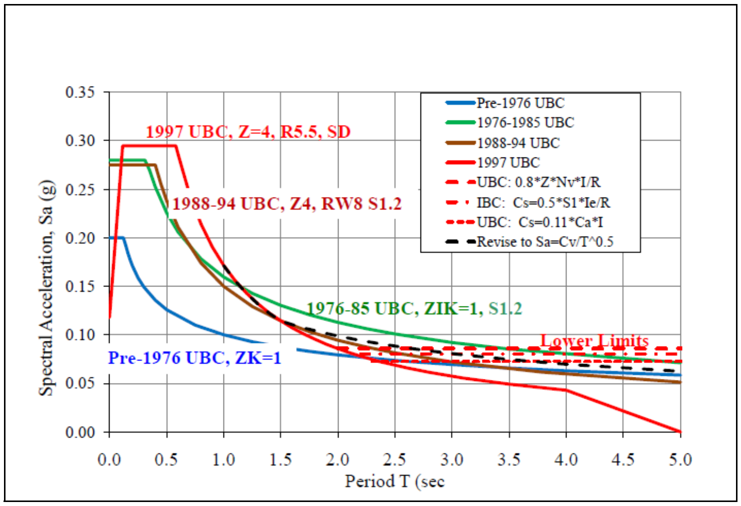

Figure 1. Comparison UBC 1973 Through UBC 1997 Design Strength Spectra (phi=1.0) for Zone 4, Soil D.

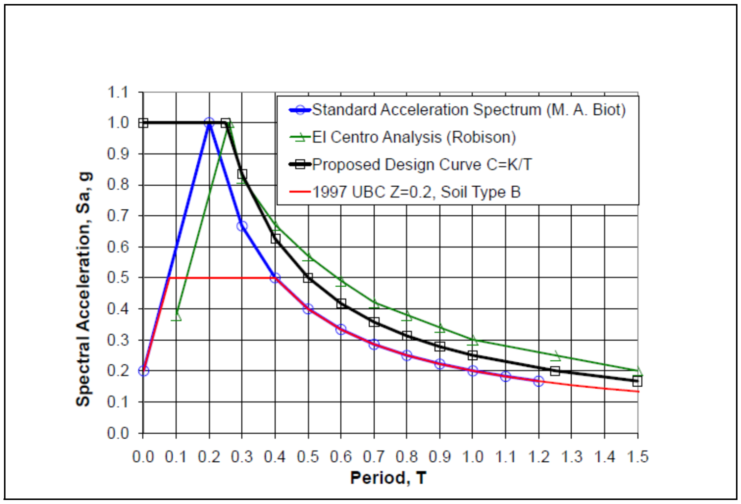

Figure 2. 1952 Joint Committee Development of Response Spectra (from Freeman 2007)

Response Spectra

In 1959, the Seismology Committee of the Structural Engineers Association of California published "Recommended Lateral Force Requirements" (generally referred to as the SEAOC bluebook) and included "Commentary" in 1960 (SEAOC 1960). Influenced by the Joint Committee (many of the members were on both committees), recommendations were proposed that were adopted for the 1961 Uniform Building Code (UBC) (ICBO 1961). The new recommended design lateral force coefficient was $C = 0.05 / T^{1/3}$ and the live loads were not included in the weight (except for a percentage in storage facilities). By using T to the one-third power, the equation could account for higher modal participation and give a larger load factor for tall buildings. In addition it avoided the need for a minimum cut-off. The maximum was set at $C = 0.10$

Over the years, the SEAOC bluebook and the UBC went through many revisions, generally influenced by some event such as the 1971 San Fernando, 1989 Loma Prieta, and 1994 Northridge earthquakes and by data relating to soil effects. The comparable curves shown in Fig. 1 have been adjusted to represent strength design response spectra and include factors representing soil classification type D.

Background to the Minimum 0.85V Provision

The code development during the 1960s and 1970 were based on dynamics and modal analysis, where the K-factor represented structural performance characteristics, the C-factor represented a response spectrum and W was the total weight of the building. The C-factor included the inverse of the fundamental period T of the building to a power of less than 1 (e.g. $T^{1/3}$). Additional factors include Z (seismic zone), I (importance of building function) and S (soil). In other words, the lateral force equation was an attempt to imitate a modal analysis with a simple single mode equivalent that was adjusted to account for higher mode effects for typical building configurations. This produced an equivalent lateral force procedure (ELFP). The standard response spectrum is assumed to have a constant acceleration portion for short period buildings and a constant velocity (function of acceleration divided by period, Sa/T) portion for long period buildings.

In the 1980s dynamic analysis provisions were added to the building codes. As an attempt to place a lower limit on the results from the dynamic analysis, the base shear of the dynamic analysis was to be normalized upward to 90% but need not exceed 100% of the base shear determined by the equivalent lateral force procedure (SEAOC 1988). Although this may have been seen as being reasonable, in other words modify the vertical distribution of forces but maintain the same base shear, there were unintended consequences. For example, assume three different buildings of similar size, but with different stiffness and weight distributions: 1. regular building with a near straight line fundamental mode shape, 2. Soft first story building with a large portion of drift in 1st story, and 3. Rigid base structure with a cantilever type mode shape.

Table 1 shows assumed effective modal mass ratios (EMMR) for each building type for three modes of vibration. These ratios can be used to calculate the base shear coefficient (V/W) from the modal spectral accelerations (Sa). The modal combination (SRSS) is shown for sample short and long period buildings. The column labeled Short T assumes all three modes are at or near the constant acceleration plateau (e.g. first mode period of 0.5 sec). The Long T column assumes the first mode (e.g. greater than 1 sec) is down the constant velocity curve at less than 40% of the plateau acceleration and the higher modes are at or near the plateau. The results show that for the short period rigid base type structure the modal analysis solution has to be increased by 21% and the regular building by 10%. What is the basis for having to increase the dynamic analysis results for these buildings? Technically, the soft story building would be expected to have the highest base shear coefficient because of the higher first mode EMMR. With the current code minimum value of 85% of the base shear (0.85V rule), more complications occur including the effects of soil factors on modal analysis, especially when applied to a non-building structure such as a tall concrete chimney.

Table 1. Relationship of SRSS Base Shear Coefficient to ELFP Base Shear Coefficient.

Building Type	**Effective Modal Mass Ratio**			**Effective SRSS Mass Ratio**		**Normalize to 0.90V**	
	1st	**2nd**	**3rd**	**Short T**	**Long T**	**Short T**	**Long T**
1.Regular	0.80	0.15	0.05	0.82	0.89	1.10	1.01
2. Soft Story	0.90	0.08	0.01	0.90	0.92	NA	NA
3. Rigid Base	0.70	0.22	0.08	0.74	0.90	1.21	NA

One of the problems can be attributed to the ELFP horizontal force distribution, which essentially forms a composite shear diagram of the multi-mode maximum responses of the story shears. In other words, it is an envelope of response shears (SEAOC 1967). If the force and shear diagrams only included the fundamental mode, the overturning moments (OTM) could be calculated directly; however, because the code shear distribution includes higher mode effects, the resulting code overturning moments (i.e. flexure) tend to be greater than those obtained by a modal analysis. Modal analysis combines forces, accelerations, shears, and bending separately. Thus, the beauty of the modal analysis procedure; it gives realistic force, shear, and OTM diagrams.

The modal analysis procedure (MAP) is linked to the design response spectrum and the ELFP is linked to the base shear coefficients (C_a and C_v). In the current codes these are linked to the same curve except C_v has some lower limits (refer to red horizontal lines in Figure 1).

The setting of lower limits on the MAP has changed over the years. Vt is the MAP base shear and V is the ELFP base shear. Examples, which to the best of my interpretation, follow:

- ATC 1978. If $V_t < V$, normalize to V. If $V_t > V$, may reduce to V.
- SEAOC 1988. If $V_t < V$, normalize to V if irregular and 0.9V if regular. If $V_t > V$, may reduce to V.
- SEAOC 1999. V_t is normalized to a portion of V. If irregular, total V; if regular and code spectra, 0.9V; and with site specific spectra, 0.8V.
- IBC 2000: If $V_t < 0.83V$, normalize to 0.83V (this comes from the inverse of 1.2 times period T). If $V_t > V$, may reduce to V.

- ASCE 2002: If $V_t < 0.85V$, normalize to 0.85V. No provisions to reduce base shear if $V_t > V$.

The ELFP base shear, V, is determined on the basis of seismic code parameter as well as limitations on the period T (i.e. the T used in the base shear equation cannot be greater than a prescribed factor times the prescribed "approximate fundamental period" T_a). One might ask where the factored V, such as 0.85V, comes from. First, it is generally recognized that the effort of a dynamic analysis should be given the benefits of a load reduction if the results indicate a reduction. However, there also seems to be a need by some that the potential reduction requires a lower limit. There also seems to be a need by some not to require the MAP base shear to be greater than the ELFP base shear. However, this urge to equalize the base shears is not consistent with the technology of dynamics of buildings. Depending on dynamic characteristics, using the same response spectrum and the same building weight W, base shears will justifiably vary, as well as the load distribution along the height of the building. In addition the base OTM will vary and is not in the same proportion as the shear. The use of the 0.85 factor may be related to the fact that the first mode EMMR for generally regular buildings is typically about 0.80. When combined with higher modes base shear ratio averages about 85% of the code base shear.

Figs. 3 and 4 are shown to illustrate the effects of variations on modal combinations and the soil site factors. In Fig. 3, a 15-story office building, the calculated fundamental period is 3.2 seconds. The 2nd and 3rd modes, with shorter periods, have substantially larger spectral accelerations than the 1st mode. With the exception of the 3rd mode on soil class E, they are essentially on the constant velocity curve. The higher accelerations somewhat compensate for the lower EMMRs of the 2nd and 3rd modes, such that the higher modes have a significant effect on the SRSS combined base shear. Thus, this example is less likely to be subject to the 0.85V rule. An exception may be due to a lower limit requirement of $0.5S_{1,}$ especially for soil class B.

In Fig. 4, a 250 foot tall chimney, the calculated fundamental period is 1.5 seconds. In this case, the 2nd and 3rd modes are capped by the constant acceleration plateaus, thus they have a lesser effect on the SRSS combination (especially for soil class E). Thus, for this example, the 0.85V rule is more likely to apply. It should be noted that in these examples the EMMR of the first mode is in the neighborhood of 0.6 for the chimney and 0.7 for the building, rather than the typical 0.8.

V is equal to C_sW. C_s is determined by the basic response spectra type equations; however there are two equations that establish lower limits. One limit is $Cs = 0.044S_{DS}I_E$ that relates to the old 3% lower limit for allowable stress design (the 0.044 represents the increase due to strength design and the S_{DS} is typically 1.0g for California). The second limit is $Cs = 0.5S_1I_E/R$ which applies if S_1 is 0.6g or greater (typical of California sites). My understanding is that this lower limit was placed in the code to account for the potential of strong velocity pulses or a lurch in regions of very high seismicity. It would seem that this issue should be addressed within the response spectra rather by limiting the base shear. As one studies the effects of the 0.85V rule on modal analysis of buildings, it becomes obvious that there are inconsistencies and some irrationality. However, I would like to leave further discussion to another day and now discuss the effects of the 0.85V rule to the topic of this paper: Tall chimneys.

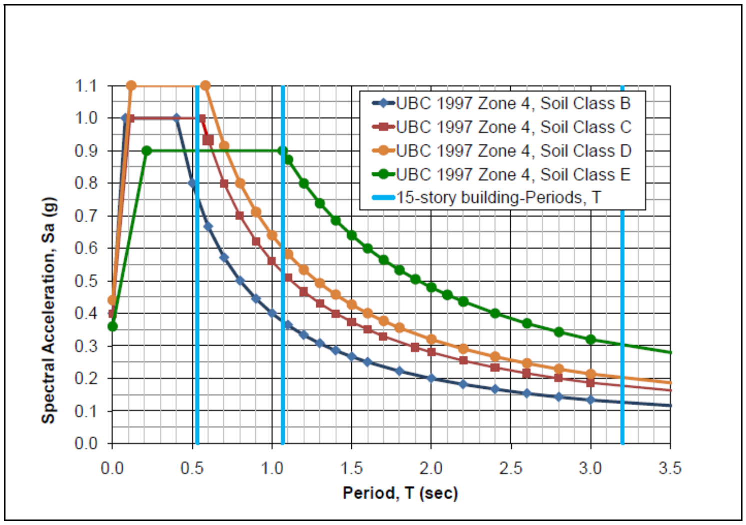

Figure 3. 1997 UBC Zone 4 Response Spectra with Modal Periods of a 15-Story Building.

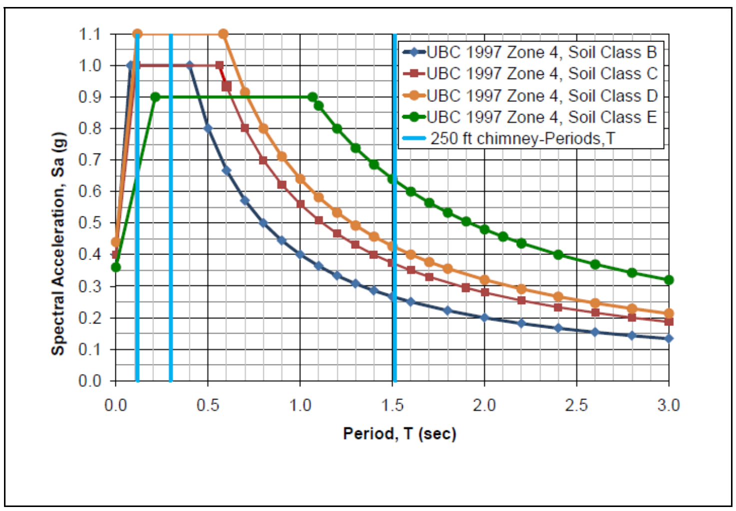

Figure 4. 1997 UBC Zone 4 Response Spectra with Modal Periods of a 250-foot Chimney

Tall Reinforced Concrete Chimneys (ACI Committee 307)

The ACI 307-98 (ACI 1998) was developed on the basis of earlier design codes that were based on earthquake ground motion with 10% probability of being exceeded in 50 yrs (or with an average return period of 475 years). The IBC-2000 and ASCE 7-02 are based on earthquake ground motion with 2% probability of being exceeded in 50 yrs (or with an average return period of 2475 years). To conform to the new seismic standards, Committee ACI 307 during the past few years has been developing an updated document that has now been published as ACI 307-08. A draft of ACI 307 considered an R=2, but this was changed to 1.5 in the final document. However, consideration is being given to going back to R=2, but with an over-strength factor of 1.5 at openings. The ACI 307 provisions do not require the seismic base shear to be scaled to the minimum values set forth in ASCE 7-02 (i.e. 0.85V not required). A great effort went into the development of ACI 307-08 with contributions of committee members well versed in the design of tall reinforced concrete chimneys. Design examples were evaluated to determine the effects of the new provisions on real chimneys. Research by others was also reviewed. It was concluded that the seismic provisions will produce safe structures, well within the intent of the codes. Performance based reviews indicated that the chimneys would generally remain elastic for the design spectrum and would be subjected to only low ductility demands during the full MCE spectrum (i.e. without the 2/3 reduction). An ASCE 7 committee, after review and discussion, generally accepted the ACI 307-08 provisions for an upcoming ASCE 7 edition with one exception: The inclusion of the 0.85V rule. As discussed earlier, it is my opinion that this requirement is not justified and possibly counterproductive. If there is a need to establish minimum bounds for the modal analysis method, they should reasonable and rational.

Tall reinforced concrete chimneys are unique compared to most structures. They are tapered in diameter, weight, and stiffness. Thus, they generally have a relatively low EMMR for the first mode of vibration (e.g. less than 0.60 rather than a typical value of 0.80 for buildings). This results in relatively larger participation of the higher modes in the total dynamic response to earthquake ground motion. Design periods are based on gross concrete sections and are generally shorter than code period, T_a. The lateral load capacity is governed by flexure. Shear does not govern. The only potential areas that could be shear critical would be around large openings, but these are well protected by good detailing requirements.

Chimneys designed in accordance with ACI 307-08, as illustrated by studies of sample designs, generally exceed ASCE 7-02 lower limits expressed by the 0.85V rule. Exceptions appear to be for very tall chimneys founded on rock-like soil class B and chimneys founded on soil class E. Table 2 summarizes results of study of 10 chimneys and one building. As discussed earlier, the 0.85V rule can be broken down to three categories: (1) Basic base shear equation (C_vW/T), (2) the minimum $0.044S_{DS}$ equation, and (3) the minimum $0.5S_1$ equation. Table 2 gives the ratio of 0.85V to V_t, which if greater than 1.0 is used as a multiplier to increase the MAP. It is the basic base shear equation that is responsible for multipliers in soil E. The $0.044S_{DS}$ equation does not govern because of the low R value used for chimneys. The 0.5S1 equation governs those marked by an asterisk (*).

The need to increase the soil E examples is not justifiable. The cause of the increases is due to the lower influences of the higher modes to the base shear. The reason the others are not subject to the increase is because of the high influences of the higher modes. However, all cases are consistent with the procedures of modal analysis. To change these relationships because of the arbitrariness of 0.85V rule seems senseless.

The case of the 0.5 S_1 limitations is a different story. First, it only applies if S_1 is equal or greater than 0.6g. So, if S_1 was equal to 0.59g instead of 0.60g, the increases would not apply. Second, the limitation is to be applied to the response spectrum (or C_v curve) to account for the potential pulse at very high seismic sites influenced by near source conditions. If these limitations are applicable to the site (a topic beyond the scope of this paper), they should be applied directly to the response spectrum that will only affect the first mode and not the higher modes. In other words, they should not be applied to the SRSS solution. For chimneys, if site specific spectra are used they should include these near source effects. I understand that new seismicity maps have been developed for the next edition of ASCE 7 that would take these concerns into account. For example response spectra shapes will vary for high seismicity sites such that that the denominator in the long period range will not be T, but equivalent to a power of T less than one. The effects of applying the 0.5 S_1 limitation to the modal analysis spectrum for the chimneys in Table 2 gave some interesting results. The base shears resulting from applying $0.5S_1$ to the first mode for chimneys #11 and #12 were less than those using the 0.85V rule; but, the base shears for chimneys #4 through #9 were greater. In addition, the resulting base OTMs were substantially different. Again, the 0.85V rule is not rational.

. Table 2. Comparing Base Shear Coefficient of 10 Sample Chimneys and a Building.

Sample Chimney	1st Mode Period	Height, feet	Normalization Multiplier 0.85V/Vt			
			Soil B	Soil C	Soil D	Soil E
Sample Chimney	1.35	460	NA	NA	NA	1.14
Chimney #1	1.52	250	NA	NA	1.03	1.26
Chimney #2	1.35	275	NA	1.01	1.10	1.23
Chimney #4	2.05	375	NA	NA	NA	1.15
Chimney #5	2.23	425	NA	NA	NA	1.12
Chimney #6	1.52	485	NA	NA	NA	NA
Chimney #8	2.32	545	1.16*	NA	NA	1.09
Chimney #9	2.56	613	1.11*	NA	NA	1.05
Chimney #11	3.64	773	1.58*	1.32*	1.18*	NA
Chimney #12	3.57	978	1.58*	1.34*	1.20*	NA
Building Example	3.20	150	2.06*	1.68*	1.50*	1.17

* designates normalization governed by $0.5S_1$ limitation that applies to sites where S_1 is equal or greater than 0.6g. If S_1 is less than 0.6g the normalization is not required.

Conclusions

It has been concluded by the author that the 0.85V rule is not a valid limitation for the modal analysis procedures for chimneys, as well as other structures. In the author's opinion, based on experience in seismic code development, the 0.85V rule as well as the $0.5S_1$ limitation was put into the codes as interim measures to accommodate legitimate concerns. As often happens, these interim provisions become somewhat permanent and are never re-evaluated. Although this paper addresses chimneys in particular, it is recommended that both the 0.85V rule and the $0.5S_1$ limitation issues should be revisited and revised to be consistent with the modal analysis procedures. For example, in lieu of the 0.5S1 cut off, revising the Cv/T equation to $Cv/T^{1/2}$ for periods over 1.5 seconds might be considered (Refer to Fig. 1).

For chimneys, it is recommended that 0.85V minimum base shear requirement in the latest ASCE 7, as applied to ACI 307-08, be deleted.

For the ACI Committee 307, it is recommended that the $0.5S_1$ limitation be reviewed for its effects on chimneys. As noted, this provision only applies if S_1 is equal or greater than 0.60g. If a chimney is designed on the basis of a site specific spectrum, the effects of high seismicity near source conditions should have been included. If a standard spectrum is used, provisions should be made to account for these potential site effects.

This paper solely presents the opinions of the author based on his understanding and interpretation of the issues. Comments by others are welcome.

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