# POORLY WORDED, ILL-CONCEIVED, AND UNNECESSARY CODE PROVISIONS

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## Abstract

Both the Uniform Building Code (UBC) and the International Building Code (IBC) -- to some degree or another -- have poorly worded provisions that are unnecessarily complex, are ambiguous, are not justifiable, and/or are overly restrictive. This paper presents just a few of these provisions, the reported justification for these provisions, and suggestions to simplify and improve the codes. Provisions discussed in this paper include the redundancy/reliability factor, the vertical earthquake, minimum base shear and drift limits, and accidental torsion.

## Introduction

All building codes have poorly worded provisions that are unnecessarily complex, are ambiguous, are not justifiable, or are overly restrictive. The Uniform Building Code (UBC) and the International Building Code (IBC) are no exception -- having numerous provisions that could be modified, improved, or even eliminated to greatly simplify and clarify the intent and wording. This paper presents just a few of these provisions, the reported justification for these provisions, and suggestions to simplify and improve the codes. Provisions in the code that could be improved or deleted include the redundancy/reliability factor, the vertical earthquake, minimum base shear and drift limits, and accidental torsion. Many of the examples and some of the text in this paper comes from other papers (Searer, 2000, and Searer and Freeman, 2002), however, additional backup and justification have been added to each section.

## Redundancy/Reliability Factor

One of the best examples of how ill-conceived provisions can unnecessarily complicate a code was the incorporation of the redundancy/reliability factor, rho ($\rho$), into the 1997 Uniform Building Code. Redundancy has long been thought beneficial to the performance of structures under extreme loads (including seismic loads). According to the 1996 Blue Book (SEAOC, 1996), the objectives of having a redundant lateral force resisting system are to:

> "1) Create a system that will have its inelastic behavior distributed nearly uniformly throughout the plan and elevation of the system; [and]
>
> "2) Have such a degree of redundancy that softening or failure of a particular element will result in redistribution of the load to the associated redundant elements, without the possibility of collapse."

Prior to the 1994 Northridge earthquake, researchers had been discussing the use of redundancy factors based on the number of lines of resistance in a given structure; however, numerous disclaimers were included with these proposals that stated that the proposed factors had "no technical basis" and were "not intended for implementation in seismic codes or guidelines" (ATC-19 and ATC-34, 1995). Despite these warnings, a redundancy factor was added to the 1997 UBC.

The 1994 Northridge earthquake was a catalyst for many code changes, including the addition of the redundancy/reliability factor. The discovery of fractured connections in special moment resisting steel frames led some to believe that a lack of redundancy contributed to the fractures. Similarly, the damage to and/or collapse of a number of precast concrete parking garages led some to believe that a lack of redundancy in the garages placed high demands on the diaphragms, which then failed. The rho factor was added to the 1997 UBC to help "encourage" the design engineer to increase the number of lateral force resisting elements to a "reasonable" level (Structural Engineers Association of California 1999). However, when viewed with the clarity of hindsight, both of these justifications for incorporation of rho into the building code were severely flawed.

**Steel Moment Frames:** During the development of the rho factor, anecdotal evidence was discussed that indicated that some modern steel special moment resisting frames were much less redundant than previous moment frames designed and built "in the good old days" when every steel structure had a full moment frame. Thus, during committee deliberations, an $R_w$ of 12 (or R of 8.5) was deemed to be inappropriate for designing structures in which only a small percentage of the columns were designed to resist lateral forces. However, a brief review of prior building codes repudiates this claim. While the author agrees that in the 1960's, a common design philosophy was to use every column or nearly every column as part of the lateral force resisting system, the truth is that in the 1960's, steel structures were merely *assumed* to be ductile. In the 1961 UBC, for example, the requirements for welded connections were less than half a page long. Likewise, riveted connections had few, if any special requirements; and requirements for strong column/weak beam behavior or requirements that beam-column connections fail in bending and not shear were nonexistent. Thus, in the early days of seismic codes, steel moment frames were more like ordinary moment frames rather than the special moment resisting frames of today. Coincident with incremental restrictions on the design and construction of special steel moment frames, economic pressures reportedly forced designers to design fewer and fewer lateral force resisting elements, an approach that was justified under the assumption that the newer designs were more and more ductile. So, while the author agrees that the Northridge earthquake demonstrated that the designs were not as ductile as previously thought, the problems had to do with detailing and quality of construction, rather than with the number of lateral force resisting elements, as discussed below. It is also important to recognize that while steel special moment frames are rewarded with very high R and $R_w$ factors, these factors generally do not govern the design, since drift limits tend to control the design of moment frame structures, and the *effective* R and $R_w$ factors for these structures tend to be significantly lower.

During the author's investigation of rho, a number of engineers also stated that rho was specifically added to preclude a certain design firm from southern California from designing tall buildings with only a few moment frame bays in each direction. Indeed, two tall buildings reportedly designed by this firm with two-bay perimeter moment frames experienced a number of fractures during the Northridge earthquake; however, even these fractures did not make the structures unstable and the buildings remained occupied during the repairs.

Due to the concern generated by unanticipated fractures in their connections, special moment resisting frames were severely penalized by the rho factor in two unique ways. Firstly, the rho formulation for moment frames requires that the shear from adjacent columns in any given frame be added together (and multiplied by 70% if the columns are not located at the "ends" of the moment frame). This formulation produces a rho that is 40% larger for moment frames than rho for other lateral force resisting systems. Therefore, moment frames require approximately 40% more lateral force resisting elements than other lateral force resisting systems to avoid the rho penalty. This overconservatism is doubly compounded by the limitations for special moment frames, which are arbitrarily limited to a rho of 1.25 or less in the 1997 UBC and 1.1 in some cases in the 2000 IBC; frames with rho greater than this limit must be redesigned.

Several years after the Northridge earthquake, in-depth studies of the welded moment connection problems revealed that redundancy or lack of redundancy had no discernible relationship to the likelihood of fractures (FEMA, 2000) and that much of the so-called damage originally ascribed to steel moment frames was actually original construction defect (Paret, 2000). In fact, not a single steel moment resisting frame structure collapsed during the Northridge or Loma Prieta earthquakes despite some structures having a significant percentage of their connections broken. This evidence points to the inherent reliability and redundancy of steel moment frames, even those with pre-Northridge detailing. Since the Northridge earthquake, new technologies such as reduced-beam sections, bottom haunch connections, and bolted bracket connections have greatly reduced the likelihood of weld failures in steel moment frames. The use of ductile weld metal, the increased awareness of the importance of special inspection of welds, and the near-fault base shear increase have further improved the reliability of steel moment frames. Given the fact that the detailing requirements for welded steel moment frame connections are now among the most studied and the most reliable structural systems available, the author does not understand why the rho formulation in current codes significantly penalizes (and thus discourages) welded steel special moment frames. Though the author recognizes that redundancy is a desirable characteristic in all structures, it appears that this impetus for the development of the rho factor is no longer valid.

**Precast Parking Garages**: Data from the Northridge earthquake also suggested that the pre-Northridge design of parking garages -- particularly precast concrete parking garages -- was lacking, and that parking garages -- as they were being designed -- were in some way less reliable and less redundant than most structures. However, further analysis of the failures indicates that a few apparent weaknesses in the garages led to the failures, including the presence of short or “captive” columns, lack of appropriate deformation compatibility, and lack of sufficient collector elements. To date, no parking structures have been identified where implementation of rho would have identified the problem or somehow prevented a collapse. Ironically, although rho was reportedly added to address relative weakness and poor performance in diaphragms (Hamburger, 2000), the final formulation generally did not affect the design of diaphragms; thus rho was ineffective in improving design of diaphragms (if any need for improvement existed). Furthermore, combined with the near-field base shear increases and the increased deformation compatibility and collector design force requirements, the new requirements for precast concrete gravity systems will produce significantly more reliable designs, and any purported need for rho is significantly curtailed, even for structures with precast concrete gravity systems.

**Haste Makes Waste**: After interviewing a number of engineers who were responsible for the development of rho, after testing rho with sample buildings, and after receiving feedback from engineers who have used rho in design, it is clear that rho was adopted into the building code without sufficient testing or backup. No one involved in the development of rho can remember on which buildings, if any, they tested rho. Code committee members reported that a number of formulas were proposed -- all of which seemed not to work -- and that as the deadline for code changes grew closer, no formulas had been found that quantified redundancy accurately. Reportedly, just before code change proposals were due, the formula used in the 1997 UBC was developed. At least three well-known structural engineers (Freeman, 1995, McClure, 2000, and Englekirk, 2000) protested inclusion of such a flawed formula in the code, but their concerns were overridden. Furthermore, rather than limit rho to the two structural systems that were unjustly assumed to have had redundancy problems during the 1994 Northridge earthquake (and were later found not to have problems with redundancy), the code committee elected to extend rho to all structural systems, further compounding the mistake. As a result of the rush to amend the building code without proper justification, a number of problems arose. The first seems immediately apparent to anyone checking the stability of the formula: since rho had no empirical or theoretical basis, there were a number of structural configurations for which the formula didn’t work or provided illogical, counterintuitive, or nonsensical results (Figures 1 through 4). Numerical backup for the results are provided in prior papers (Searer, 2000, and Searer and Freeman, 2002). As the examples show, the formulations in the 1997 UBC, the 2000 IBC, or ASCE 7-05 do not agree on whether the example structures are redundant or reliable, leading one to conclude that perhaps none of the formulations are meaningful.

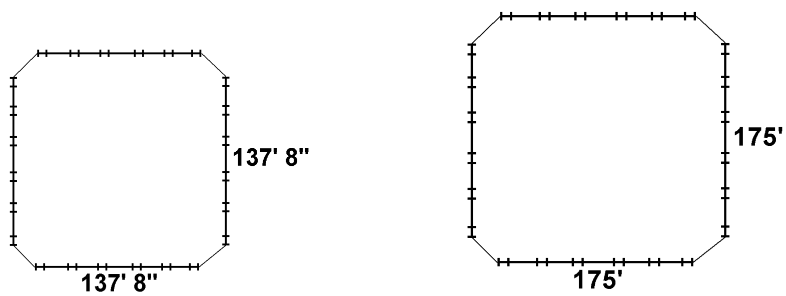


Figure 1: Plan view of two special steel moment resisting frames. Despite the fact that both structures have an equal number and percentage of lateral force resisting elements, the moment frame on the left is deemed redundant and reliable by the 1997 UBC formulation and the moment frame on the right is deemed significantly nonredundant and unreliable and must be designed for a base shear coefficient that is 65% greater than for the structure on the left. According to the 2000 IBC, the structure on the right is so unreliable and nonredundant that it must be completely redesigned.

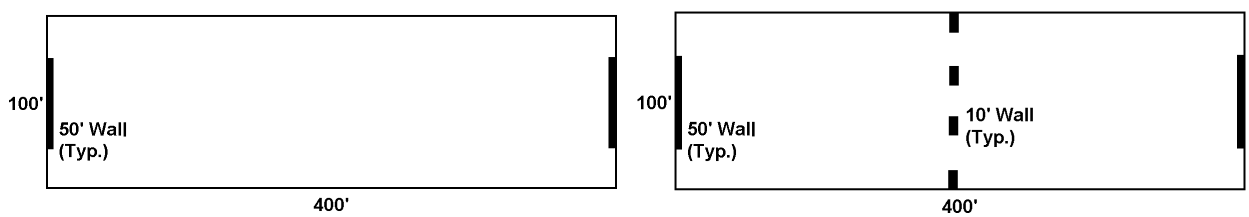


Figure 2: Plan view of two tilt-up structures with flexible wood diaphragms. The structure on the left is considered redundant and reliable. Nonsensically, the structure on the right, which has a larger number of walls, greater wall length, and an interior line of resistance, is considered significantly nonredundant and unreliable and is penalized by an increase in its base shear coefficient of 20%. According to ASCE 7-05, both structures are sufficiently redundant, despite the fact that the structure on the left has a very long, very slender diaphragm and does not necessarily appear to be a "good" design.

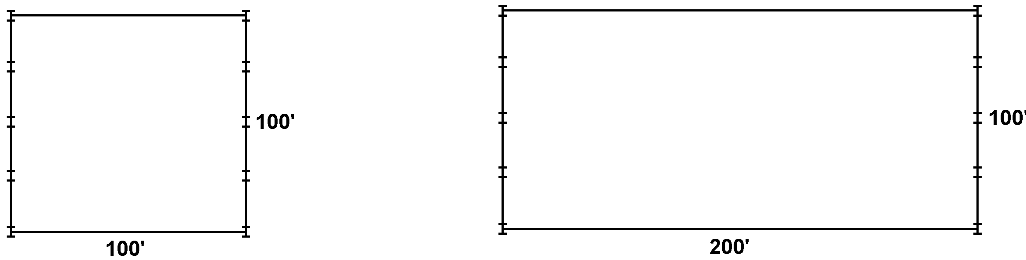


Figure 3: Plan view of two steel moment frame structures with complete perimeter moment frames (frames are only shown in one orthogonal direction). According to the 1997 UBC and the 2000 IBC, the structure on the left is redundant and reliable and the structure on the right is so nonredundant and unreliable that it is not allowed to be built and must be redesigned. According to ASCE 7-05, both structures are twice as redundant as they need to be to qualify as redundant.

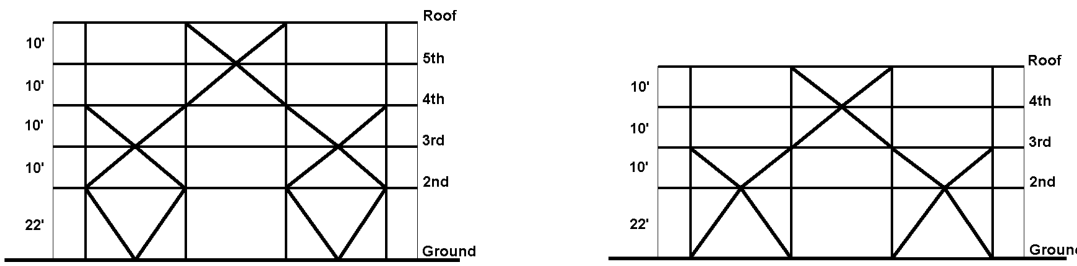


Figure 4: Elevations of two steel braced frame structures that are square in plan. According to the 1997 UBC, the taller structure on the left is redundant and reliable and the shorter structure on the right is so nonredundant and unreliable that it is penalized with a 37% increase in its design base shear coefficient. According to the 2000 IBC, both structures are nonredundant and unreliable. As for ASCE 7-05, the structures would have to be structurally analyzed to determine whether or not removal of any single brace would reduce any story strength by more than 33% or result in an extreme torsional irregularity.

**Implementation in Design**: As the 1997 UBC came into use, practicing engineers reported that rho actually discouraged them from designing redundant structures. Tilt-up designers reported that rho discouraged the use of interior lines of resistance (Deardorff, 2000). Designers of steel moment frames reported that rho prevented them from designing structures that were clearly redundant and reliable and encouraged them to use less redundant and less reliable structural systems and designs (Behshid, 1996). Designers of wood structures reported that the results from rho unfairly and irrationally penalized wood structures (APA, 2000, and Greenlaw, 2000). For many of the reasons described above, a study of the rho factor by the Structural Engineers Association of Northern California (SEAONC) concluded that the current formulation of rho is “deeply flawed” (SEAONC, 2001). Despite the numerous and well-documented problems with rho, engineers in charge of code development have steadfastly refused to delete rho from the code. However, since the formula was simply “made-up,” it was also impossible to propose a rational fix for the formula that would mitigate the documented problems. For nearly a decade now, the “deeply flawed” rho has been incorporated into the California Building Code and apparently will remain in the code until the IBC is eventually adopted.

In a recent peer-review project, the author reviewed the structural design of two five-story concrete residential structures. The author found that the 1997 formulation of rho identified both structures as being nonredundant and unreliable and penalized the design with an approximately 30% increase in base shear. Conversely, although the author has not performed a rigorous analysis with the newest rho formulation, ASCE 7-05 appears to identify the structures as sufficiently redundant due to the presence of perimeter shear walls. Neither redundancy/reliability formulation identifies the real problems with the design, including failure to provide ductile detailing in the columns, failure to provide required boundary elements in the shear walls, failure to provide sufficient strength for columns and beams that support discontinuous walls, failure to consider punching shear in the flat slabs due to deformation compatibility, failure to provide sufficient foundation elements to resist overturning, failure to properly estimate the weight of the structure, failure to provide chord steel and collector elements, failure to provide reinforcing indicated by the engineer's own computer analysis as being required, and failure to check the computer output. When questioned how so many errors could be present in the design, the engineer responded that the computer program he used is essentially a "black box" and that it would be impossible to check such a sophisticated computer model, which, as it turned out, was an ETABS, rigid diaphragm, linear, response spectrum analysis! In this case, no formulation of rho could possibly address the real problems in the design, much less address the root cause of the poor design -- that of poor engineering.

**Recent Developments**: The recently published *Minimum Design Loads for Buildings and Other Structures* (ASCE/SEI 7-05), the basis for the 2006 IBC, now contains a different formula for rho. The new formulation for rho assumes that a structure is nonredundant and penalizes the structure with a 30% increase in design base shear unless it can be shown that elimination of a brace, beam, wall, wall pier, or cantilever column will not result in more than a 33% reduction in story strength, or cause an extreme torsional irregularity. Alternately, if the structure has two bays of moment frames or shear walls on each side, the structure is also awarded a rho of 1.0. *Ironically, this formulation penalizes neither the two-bay moment frame designs nor the diaphragm designs that were the reported (but incorrect) impetus for putting the rho factor into the code in the first place.* While the author believes that this formulation is more rational and more defensible than the formulation in the 1997 UBC, there are no "lessons learned" from past earthquake that demonstrate that even this formulation will preclude catastrophic failures of the lateral force resisting system of any building. The author notes that the term "story strength" is not defined by ASCE 7-05 and that it is not clear how global overturning in a model where a wall, wall pier, or cantilever column has been completely removed is to be accommodated. The author suspects that as the new formulation is tested in real designs, implementation problems with this formulation will also be discovered. While the author agrees with the need for redundancy in design, the resultant chaos of trying to quantify redundancy has severely complicated and worsened the building code.

**Recommendations**: The 1997 UBC rho formulation is so clearly flawed and ill-conditioned that it should be eliminated from current code. The ASCE 7-05 rho formulation needs more study to determine whether or not it actually encourages more redundant structures.

## Vertical Earthquake

Another example of the unnecessarily complication of a building code was the inclusion of vertical earthquake forces into required load combinations.

**Background:** Strength design requirements based on statistics and probability were developed for both concrete and steel to produce more "rational" designs that are slightly more efficient than designs produced using the Allowable Stress Design (ASD) procedures. Historically, older versions of the UBC contained a load factor of 1.4 for dead load (D), when combined with live (L) and earthquake lateral forces (E). However, strength design requirements (such as Equation 1) typically require a load factor of only 1.2 for dead load when combined with earthquake forces, since the dead load is relatively well known compared to other loads.

$$1.2D + 1.0E + 0.5L \qquad (1)$$

In the conversion of the UBC to the IBC, SEAOC was reluctant to adopt a dead load factor of 1.2 in lieu of the historic value of 1.4, since this would result in a reduction of the design level forces compared to previous codes. For the express purpose of resolving this issue (SEAOC, 1999), SEAOC adopted a *vertical* earthquake component, $E_v$ (Equation 2), that was added to the *horizontal* earthquake component, $E_h$, to produce E (Equation 3), the earthquake design force, despite the fact that after extensive review of data from the Northridge earthquake, the consensus of SEAOC was that explicit consideration of vertical ground motion was not justified (SEAOC, 1996). It is important to note that the addition of a vertical earthquake component was not due to any evidence that vertical ground accelerations contributed to or caused any failures of structures in previous earthquakes; the vertical earthquake component was added merely to maintain parity with older provisions of the code and to maintain parity between ASD and strength design provisions (SEAOC, 1999).

$$E_v = (0.5C_aI)D \tag{2}$$

and

$$E = \rho E_h + E_v \tag{3}$$

In high seismic zones where $C_a$ is approximately 0.4, the vertical earthquake load factor adds approximately 20% more dead load, bringing the total vertical load factor for dead load back to the original 1.4; however, certain unintended consequences of this action were only discovered after the code was published. In particular, the load combination of

$$0.9D \pm 1.0E \tag{4}$$

can be expressed as two equations:

$$(0.9 + 0.5C_aI)D \ \pm\ (1.0)\rho E_h \tag{5}$$

and

$$(0.9 - 0.5C_aI)D \ \pm\ (1.0)\rho E_h \tag{6}$$

Whereas the original "±" sign in the load combination formula (Equation 4) merely indicated that earthquake forces for a given axis should be analyzed in both the positive and negative horizontal direction, adding the vertical component to the horizontal component created a *vector* and caused the vertical component to become subject to the "±" sign in the load combination formula.

In the case of Equation 6, where $C_a$ is approximately 0.4 (or $S_{DS}$ in the IBC is approximately 1.0), the vertical earthquake load factor *subtracts* approximately 20% dead load, thus reducing the total gravity load available to resist uplift to only 70% of dead load. In near-fault areas, where $C_a$ can be as high as 0.6, the available vertical load that resists overturning decreases to 60%. When practitioners began using the 1997 UBC, they quickly found that the addition of a vertical earthquake had increased the number and complexity of the load combinations and that strength designs governed by overturning and uplift that had worked under previous codes and would still work under the ASD methodology – which doesn't have a vertical earthquake – were suddenly deemed to be unstable (Zsutty, 2000). The unintended decrease in the available dead load resistance also combines in further unforeseen and unintended ways with the new strength lateral design forces (and corresponding overturning demands), which are larger than the traditional ASD forces by a factor of 1.4. Use of Equation 6 results in a net decrease of approximately 20% to 30% of nominal overturning resistance in regions of high seismicity, resulting in unjustified wholesale changes to overturning design as well as the design of vertical and lateral force resisting elements. Thus, the whole purpose of strength design was subverted; not only does the strength design require more computational effort (and a correspondingly increased opportunity for the engineer to make mistakes), but using strength design in conjunction with the vertical earthquake can result in a substantially less efficient design, without any demonstrable benefit.

Note that while at first blush, it might appear inconsistent to rely on 90% of the dead load to resist overturning when lateral design loads are significantly reduced from the elastic demands, this approach makes sense for a variety of reasons, as described below.

1) To the author's knowledge, buildings do not tend to fail in an upwards direction. While a structure might experience transient vertical accelerations that reduce the net downward resultant loads, the overall upwards accelerations typically do not come close to overcoming gravity even for an instant, and within a very brief time, the vertical accelerations have reversed and add to the overturning resistance.

2) With the exception of single-story structures, assuming that there is no live load on the structure at the time of the earthquake is extremely conservative and generally not correct.

3) In order to accelerate portions of the building rapidly upward (i.e. overcome their dead load resistance), the upwards forces must exceed gravity by a significant amount; thus the resisting weight is a factor greater -- and likely significantly greater -- than just the dead load.

4) Even if overturning of an individual element overcomes the dead load tributary to it, if the deformations are significant enough, additional dead load will be mobilized through large deformations and catenary action.

**Recent Developments**: ASCE 7-05, in an apparent effort to make allowable stress design for overturning and uplift equally as overconservative as strength design, decreases the available overturning and uplift resistance for allowable stress design even more, as shown in Equation 7.

$$(0.6 - 0.14S_{DS})D + 0.7\rho Q_E \tag{7}$$

Furthermore, even when the *maximum* expected earthquake forces are considered (Equations 8 and 9 for strength design and allowable stress design, respectively), dead load resistance is still severely underestimated.

$$(0.9 - 0.2S_{DS})D + \Omega Q_E + 1.6H \tag{8}$$

$$(0.6 - 0.14S_{DS})D + 0.7\Omega Q_E + H \tag{9}$$

Again, these changes appear to have been made without the benefit of "lessons learned" from prior earthquakes or any documented evidence that use of traditional factors of safety against overturning and uplift in prior codes resulted in significant life safety hazards. While the author believes that the problems related to strength design and overturning need significant further study, lumping conservatism upon conservatism is not a rational approach to the problem.

**Recommendations**: In the case of Equation 1, Zsutty (1999) has proposed restructuring the load cases so that the vertical earthquake, which is merely an amplification of dead load, is separated from the horizontal earthquake force, thus simplifying the load cases involving seismic forces.

$$(1.2 + 0.5\ C_a I)\ D \pm (1.0)\rho E_h + (f_1)\ L + (f_2)\ S \tag{10}$$

If the engineering profession believes that it is imperative to have a vertical earthquake component in the design of typical structures – despite the fact that vertical earthquake components have never been shown to contribute to or cause failures of structures – then this proposal is acceptable. However, in the author's opinion, a better solution would be to further simplify the process, eliminating the vertical earthquake component (and rho) as shown in Equations 11 and 12.

$$(1.2)\ D \pm (1.0)\ E_h + (f_1)\ L + (f_2)\ S \qquad (11)$$

and

$$0.9D \pm 1.0\ E_h \qquad (12)$$

## Drift Limits

Drift limits have historically been difficult to justify and have gotten worse in that respect as codes have become more complicated. The 1997 UBC requires that story drift be limited to 0.025 for short period structures and 0.020 for long period structures; these or similar limits have been in place since the 1976 UBC. ASCE 7-05 requires that story drift be limited based on the type and use of the structure. The intent of both codes was to limit the interstory drift to a reasonable value, beyond which it was thought that the structure might experience loss of vertical stability. The UBC also allows these limits to be exceeded, provided that the greater drift can be tolerated by both structural elements and nonstructural elements that could affect life-safety. For the UBC, it is not clear why there are two limits on drift, one for short-period structures and one for long-period structures. In fact, since shorter period structures can have difficulty escaping from the constant acceleration region of the response spectrum, it could be argued that the prescriptive drift limits should be reversed, with longer period structures allowed larger drift limits.

It is also not entirely clear why drift limits are required at all; if a designer properly designs a structure to withstand the maximum expected deflections and still maintain vertical and lateral stability, then a prescribed limit to interstory drift should not be needed. However, according to the 1975 Blue Book (SEAOC, 1975), drift limits were added in the 1976 UBC to “insure structural integrity and to restrict damage to such fragile non-structural elements as glass, plaster walls, etc.” While current codes require that nonstructural elements be designed to accommodate the maximum expected movement of the structure, the larger the interstory drifts, the more difficult it becomes to properly design and detail nonstructural elements such as cladding, windows, and stairs, which are all affected by interstory drifts. Consequently, it appears that requiring that drift be limited to certain maximum values is reasonable from a damage control and falling hazard perspective.

**Controversy Surrounding Equation 30-7:** There have been significant debates and controversy surrounding UBC Equation 30-7. In the published version of the 1997 UBC, two minimum base shear equations are present (Equations 30-6 and 30-7), only one of which (Equation 30-6) is exempted from drift calculations. According to a three-page position paper published by SEAOC (Bachman *et al.*, 2001), SEAOC had also originally agreed to exempt Equation 30-7 from the drift requirements. However, due to an error, the published version of the UBC failed to exempt Equation 30-7. Out of a growing concern for near-field pulse effects on long-period structures, SEAOC then reversed its position and decided to support the use of Equation 30-7 in the UBC. Around this time, ICBO, the publisher of the UBC, realized that due to an error, Equation 30-7 had not been exempted from the drift equations and issued an erratum that exempted Equation 30-7 from the drift limits. SEAOC then issued the three-page position statement that endorsed the use of Equation 30-7 for both near-fault and non-near-fault structures. However, the justification for this position statement was a comparison of a few extremely large, near-field earthquake response spectra with the non-near-field, Soil Type C design spectrum. As it turns out, the records were also reportedly “cherry-picked” so that the many response spectra that did not have large long-period demands were not shown. The position statement concluded that since the large, near-field earthquake response spectra exceeded the non-near-field, Soil Type C design response spectrum, the use of Equation 30-7 for drift calculations was needed to control drift. However, the comparison and the conclusions were flawed, since the near-field records arguably should have been compared with a near-field response spectrum with Soil Type D, which would have increased the design spectrum size in the longer periods by a factor of more than two. Figure 5 shows the response spectra from the five earthquake records that were used in the SEAOC position paper and compares these spectra with several code design spectra.

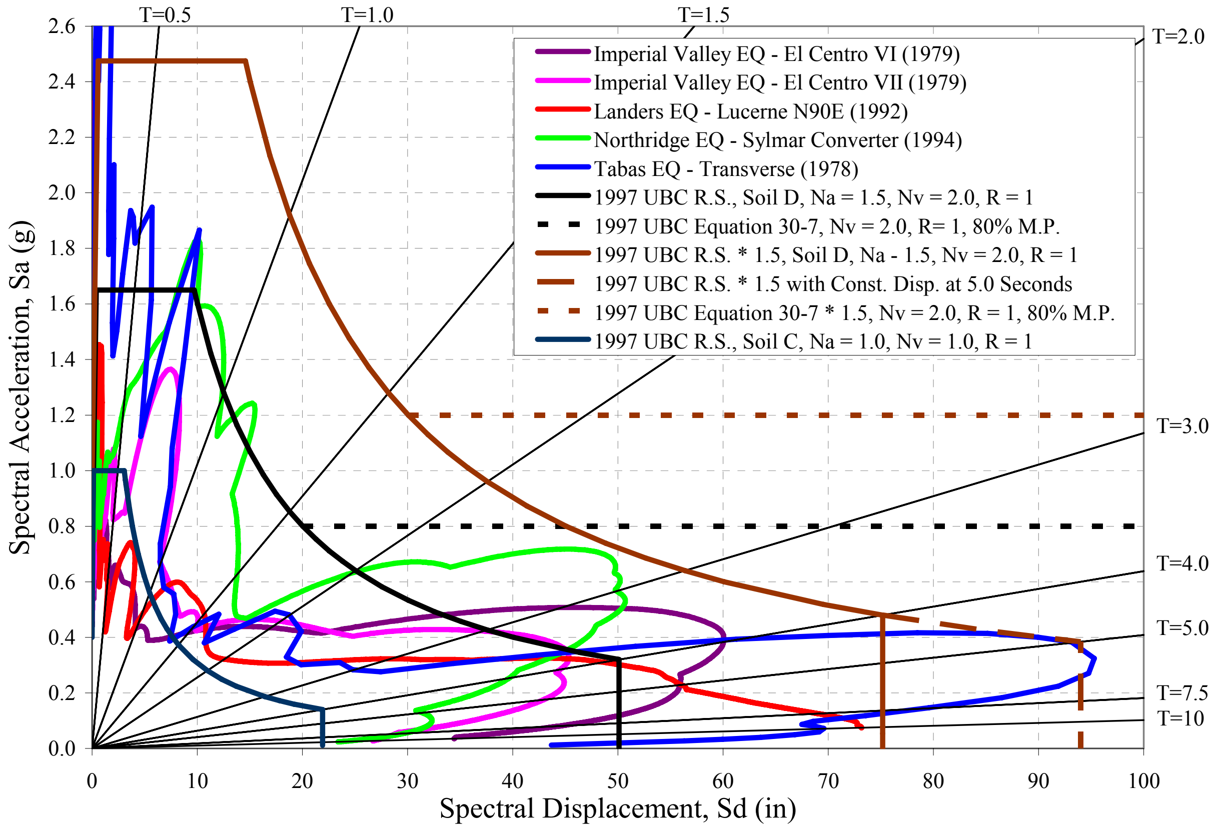


Figure 5: Comparison of five near-field earthquakes with the UBC near-field response design spectrum.

Compared to the 1997 UBC response spectra with Soil Type D, $N_V = 2.0$, and an R of 1.0, the actual earthquake response spectra exceed the design response spectrum in all three portions of the design response spectrum (i.e. constant acceleration, constant velocity, and constant displacement), which provides no specific impetus for a minimum base shear equation to address long period structures and/or the constant displacement region of the spectrum. Furthermore, when the earthquake response spectra are compared against the equivalent of the Maximum Considered Design Earthquake (1.5 times the 1997 UBC design response spectrum), the earthquakes are essentially contained within the design envelope. Assuming that the response spectrum from the 1978 Tabas earthquake is actually accurate in the long period regions, it could be argued that the constant displacement cut-off should start at 5.0 seconds instead of 4.0 seconds. Either way, it is clear from Figure 5 that the minimum base shear from Equation 30-7 (assuming Soil Type D, an R of 1.0, and 80% mass participation) greatly over-estimates any demands from these near-field earthquakes. If adjusted for the MCE event (by multiplying the demand by 1.5), the disparities between Equation 30-7 and real earthquake demands becomes even larger. Figure 6 shows another graphical comparison of Equation 30-7 with the five response spectra with the largest long-period demands, plotting spectral displacement versus period. As the figure shows, the design spectral displacements associated with Equation 30-7 trend towards infinity for long-period structures, while even the cherry-picked response spectra trend towards a constant displacement for periods exceeding 4 or 5 seconds.

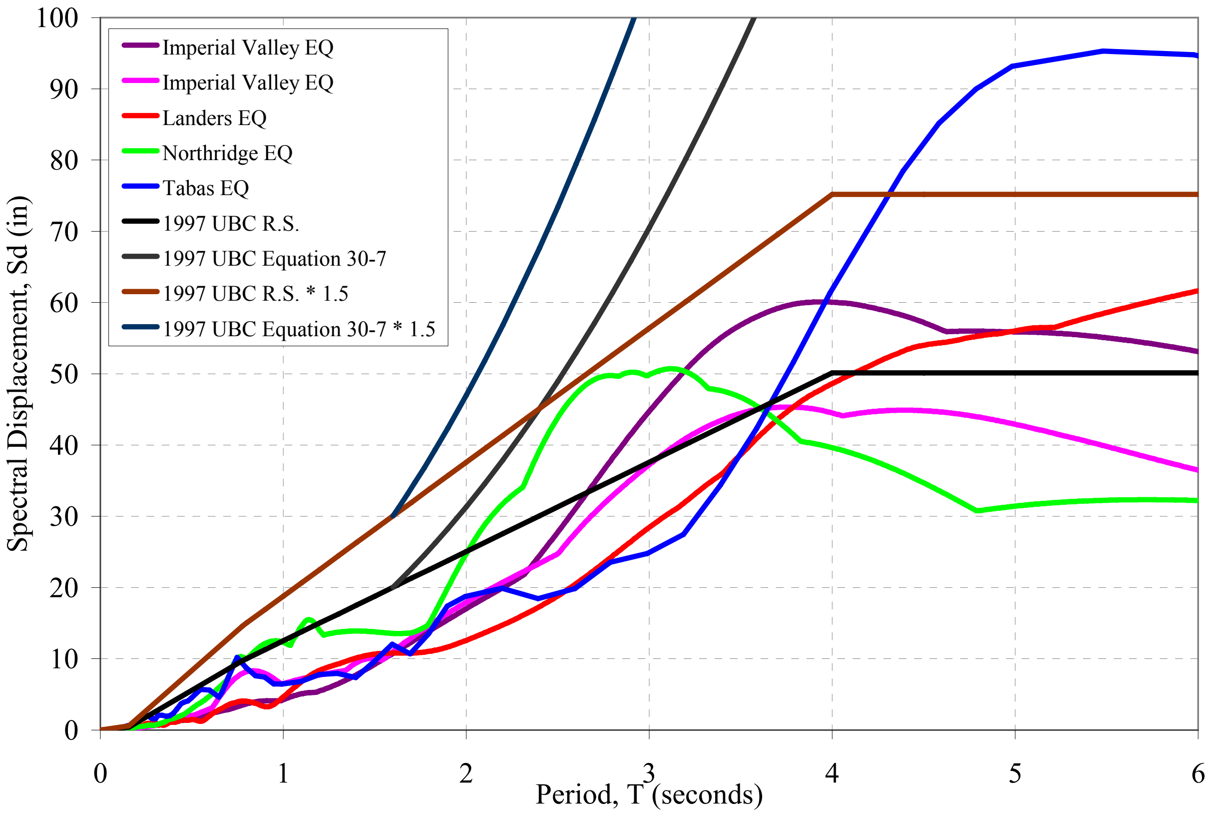


Figure 6: Comparison of five near-field earthquakes with the UBC near-field response design spectrum.

Clearly, consideration of minimum base shears in computing deflections does not appear justified at this point.

According to the 1999 Blue Book (SEAOC, 1999), the minimum base shear coefficients were added to the 1997 UBC to produce equivalence with prior codes. These prior codes had based design forces on a $1/T^{2/3}$ relationship for all structures, including long-period structures; the 1997 UBC switched to a response spectrum approach that was based on a 1/T relationship, resulting in smaller design forces for long-period structures. Older codes had used the $1/T^{2/3}$ relationship to account for modal addition. In order to ensure that newer buildings were not being designed for forces less than those of prior codes, the minimum base shear equations 30-6 and 30-7 were added to artificially increase design forces for long-period structures (SEAOC, 1999) -- despite the fact that these longer period structures typically get designed using dynamic response spectrum analyses and not the linear static method. Since the 1/T relationship is more correct than using the $1/T^{2/3}$ relationship, forcing minimum base shear coefficients on long-period structures and forcing parity between older (and less accurate) codes and current code is clearly incorrect and overconservative.

**Recent Developments**: In order to reimpose the use of Equation 30-7 for determining drift of long-period buildings, the City of Los Angeles adopted an equally counterintuitive limitation on drift, as shown in Equation 13.

$$\Delta_{\text{max allowable}} = 0.020/T^{1/3} \tag{13}$$

This has the effect of reinstating the minimum base shear equation 30-7 -- but in a non-intuitive and irrational way -- and of reinstating the illogic associated with the equation.

The recently published ASCE 7-05 also requires that minimum base shears be used to compute deflections -- again flying in the face of all known evidence that this requirement is unsupportable.

**Recommendations**: The use of a minimum base shear in the determination of drift in tall buildings is not supported by current knowledge of either earthquakes or earthquake engineering. Minimum base shear should not be used in the determination of drift in long-period structures.

## Accidental Torsion

One of the most irksome and long-standing requirements of current code is the requirement to model accidental torsion. According to the 1999 Blue Book, accidental torsion is included to account for differences between the model and the actual structure, the non-uniform distribution of dead and live loads, any eccentricities in stiffness from nonstructural elements, and torsional input from the ground (SEAOC, 1999). However, in general, as long as the structure is reasonably configured, accidental torsion tends to add only 5% to 10% to the design lateral forces for elements of the lateral force resisting system. It also dramatically increases the number of load cases that must be run.

**Recommendations**: In general, the author believes that a simple amendment to the code should be considered such that if in each orthogonal direction, the maximum plan dimension of the lateral force resisting system is at least 50% of the maximum plan dimension of the structure in that direction, then accidental torsion can be ignored. This would reward structures with well-proportioned structural systems by reducing the number of load cases required and require increased analysis and increased design forces for structures with ill-proportioned systems that may be susceptible to accidental torsion.

## Conclusions

The author has attempted to provide information and recommendations regarding four controversial code provisions that add significant complexity to the code but do not necessarily provide value equal to that added complexity. The author acknowledges that the opinions presented in this paper will likely offend some of the staunchest code-defenders -- particularly the people who wrote the provisions in the first place -- however, the paper is intended to promote discussion that has been sorely lacking in our profession in the last few years. The author would like to see a code-writing atmosphere where ill-conceived provisions and provisions that lack adequate technical basis are deleted, where poorly worded provisions are clarified, and where well-thought-out and effective code provisions are added after rigorous testing and debate.

In 2001, the Structural Engineers Association of California conducted a survey of its members, and the single most pressing issue identified by the practicing engineers who completed the survey was the need to simplify the building code (SEAOC, 2001). When engineers themselves admit that the codes are too complicated and too difficult to use, it is time for the code-writing organizations and code-writing professionals to take heed.

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