UNINTENDED CONSEQUENCES OF CODE MODIFICATION

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ABSTRACT

The code provisions for earthquake resistant design have been substantially revised in the conversion of the 1994 Uniform Building Code (UBC) through the 1997 UBC to the 2000 International Building Code (IBC). This paper examines five of the most recent changes to the code: the vertical earthquake component; the seismic design of nonstructural components and equipment; the maximum inelastic response displacement; the use of 2/3 of the maximum considered earthquake; and the reliability/redundancy factor. The apparent reasoning behind these five changes and some of the unintended consequences are discussed.

Introduction

When adopted by a state or municipality, the building code becomes a legal document, and engineers in responsible charge are generally required to follow the adopted code. Therefore, no matter what the reason, and no matter how small the change may appear, any change to a building code is a serious matter. The code provisions for earthquake resistant design have been substantially revised in the conversion of the 1994 Uniform Building Code (UBC) to the 1997 UBC and from the 1997 UBC to the 2000 International Building Code (IBC). Many of the revisions are clearly based on lessons learned from recent earthquakes, while other revisions are more obscure and less obvious. Five of the most recent changes to the code are discussed: E_v, the vertical earthquake component in the earthquake design force; F_p, the Seismic Design Force for elements of structures, nonstructural components, and equipment; Δ_M, the Maximum Inelastic Response Displacement; the use of 2/3 of the Maximum Considered Earthquake (MCE) in the IBC; and rho (ρ), the Reliability/Redundancy factor. While some engineers may object to the opinions expressed in this paper and other engineers may have stronger objections to these code changes, the goal of this paper is to discuss the apparent reasoning and some of the potential unintended consequences of these changes.

Vertical Earthquake Component

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 Eq. 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.

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$$1.2D \pm 1.0E + 0.5L \tag{1}$$

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 (Eq. 2), that was added to the *horizontal* earthquake component, E_h, to produce E (Eq. 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).

$$E_v = (0.5C_aI)D \quad \text{and} \quad E = \rho E_h + E_v \tag{2\&3}$$

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. Certain unintended consequences of this action were only discovered after the code was published. Whereas the original "±" sign in the load combination formula (Eq. 1) 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 caused the vertical component to become subject to the "±" sign in the load combination formula. Thus, Eq. 1 can be expressed as two equations:

$$(1.2 + 0.5C_aI)D \pm (1.0)\rho E_h + (0.5)L \quad \text{and} \quad (1.2 - 0.5C_aI)D \pm (1.0)\rho E_h + (0.5)L \tag{4\&5}$$

More importantly, the load combination of

$$0.9D \pm 1.0E \tag{6}$$

also can be expressed as two equations:

$$(0.9 + 0.5C_aI)D \pm (1.0)\rho E_h \quad \text{and} \quad (0.9 - 0.5C_aI)D \pm (1.0)\rho E_h \tag{7\&8}$$

Thus, the number of load combinations required to be checked for design considering earthquake forces was doubled, resulting in unintended changes to Eq. 8, as discussed below. During the review of this paper, one reviewer suggested that Eq. 5 and Eq. 7 would never govern, and thus these load combinations could be ignored. While well intentioned, the reviewer's suggestion serves to highlight the significant complexity of the code, since Eq. 5 and Eq. 7 *can govern* when forces (or moments) from dead and live load have opposite signs from each other, an uncommon but not inconceivable possibility. In the authors' opinion, the intent of the UBC is not clear. In light of the significant conflicts between SEAOC's recommendations (1996 and 1999) and the IBC provisions, justification of the vertical earthquake component appears questionable.

In the case of Eq. 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. 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 Eq. 8 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.

In the case of Eq. 1, Zsutty (1999) 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.

$$(1.2 + 0.5\, C_a I)\, D \pm (1.0)\, \rho E_h + (f_1)\, L + (f_2)\, S \qquad (9)$$

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 our opinion, a better solution would be to further simplify the process, eliminating the vertical earthquake component (and rho) as shown in Eqs. 10 and 11.

$$(1.2)\, D \pm (1.0)\, E_h + (f_1)\, L + (f_2)\, S \quad \text{and} \quad 0.9D \pm 1.0\, E_h \qquad (10\&11)$$

The authors believe that the remaining problems related to strength design and overturning need significant further study.

Design of Nonstructural Components and Equipment

The procedures for designing nonstructural components and equipment that had remained relatively untouched for decades were radically altered in the conversion from the 1994 UBC to the 1997 UBC. Not only are the design formulas more difficult to use, but the results produced by the formulas can often appear irrational. For example, consider the Allowable Stress Design (ASD) of an 800-pound piece of life-safety equipment with vibration isolators hung a few feet from the ceiling on the 29^{th} floor of an existing 30-story building, given seismic Zone 4, soil type S_D, without near-field effects.

Using Equation 32-1 of the 1997 UBC, the simplified ASD design force is 1.51 kips (1.88g). Using the more calculation-intensive Equation 32-2, a_p is 1.0, R_p is 3.0, h_x/h_r is approximately 1.0, and the ASD design force is 0.50 kips (0.63g). However, since this equipment has vibration isolators, a_p is 2.5 and R_p is 1.5, per Footnote #14 to Table O. Thus, the ASD design force is 2.5 kips (3.13g). Since this equipment will be hung from the underside of the existing floor slab, which is only 5-inches thick, Footnote #14 to Table O further doubles the connection forces for either expansion anchors or shallow anchors, resulting in a design force of 5.0 kips (6.25g). The resultant force is much higher than that produced by the less complicated equation 32-1. Even if the equipment is only mid-height in the building, the supposedly "rational" formula design force is far greater than that produced by the simple formula, and the equipment should be designed for 1.88g. *However, Footnote #14 to Table O, which is only referenced by Equation 32-2, back-references Equation 32-1, and requires a doubling of the design forces – even for Equation 32-1.*

Following this convoluted procedure, the minimum design force for the anchors for this equipment is 3.02 kips (3.77g). Since connections typically have a factor of safety of at least 3, this equipment should be good for at least 11.3g. It seems inconceivable to the authors that equipment hung less than two feet from the underside of a slab in a 30-story building would experience accelerations anywhere near this magnitude. Note that if the building had been located within 2 km of a Type A fault, the elastic design acceleration would be 5.66g, and the equipment would likely be able to withstand nearly 17g.

For comparison, the design anchorage forces in the 1994 UBC would be based on Z of 0.4, C_p of 1.5 for non-rigid equipment, and an I_p of 1.5, resulting in an ASD design force of 0.72 kips (0.90g). Thus, if designed according to the 1994 UBC, the equipment should be able to withstand 2.7g, a much more reasonable design goal.

As the above example demonstrates, the nonstructural seismic provisions in the 1997 UBC are overly complicated, are difficult to use, require a significant computational effort, and may produce results that do not appear rational. Using the raw data from previous studies (Bachman 1995), Figure 1 shows a comparison of the ratio of peak floor acceleration to peak ground acceleration from a number of instrumented buildings in a number of earthquakes. Note that except at the roof, the data indicates that a constant acceleration value over the height of the building appears to produce design values that have a reasonably consistent factor of safety (Kehoe 1998). In fact, the overwhelming majority of data points fall to the left of the heavy vertical line at a peak floor acceleration to peak ground acceleration ratio of 3.0. Except for the data at the roof, the points to the right of the vertical red line have been shown to be invalid for the purposes of studying the ratio of peak floor acceleration to peak ground acceleration (Kehoe 1998).

Therefore, the authors propose using a much more simple method to design non-structural components and equipment.

For rigid components not supported at the roof:

$$F_p = 1.4 * C_a * I_p * W_p \tag{12}$$

For rigid components supported at the roof, the design forces should be increased by an additional 50% to account for the increased accelerations experienced at the roof due to modal superposition:

$$F_p = 2.0 * C_a * I_p * W_p \qquad (13)$$

Flexible components and equipment should be designed for double these forces to account for potential amplification (i.e. resonance) that could occur due to the flexibility of the equipment.

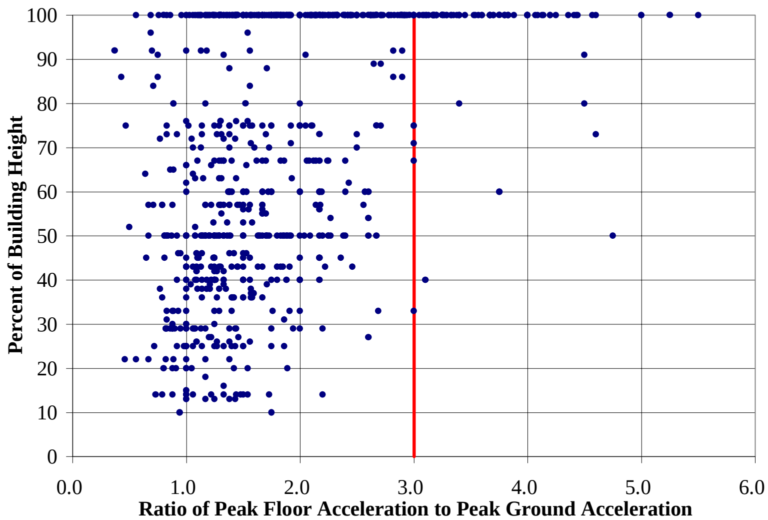

Figure 1. Historical comparison of ratio of peak floor acceleration to peak ground acceleration from instrumented buildings.

As an example, consider a rigid piece of equipment not associated with life safety, designed for a typical floor in a building on a site with a C_a of 0.4g. The ASD design force would be equal to 0.4 times W_p, slightly higher than previously required by the 1994 UBC. We can assume from Figure 1 that the *maximum* expected peak floor acceleration could be as high as three times C_a (whereas most would be less than two), or 1.2g. This is an instantaneous peak acceleration and the effective acceleration that a non-structural component could actually "see" would be smaller. Furthermore, in a large earthquake, the inelastic behavior of the building would likely reduce this acceleration significantly, and any nonstructural damage that occurs will help to decrease the maximum response of the nonstructural elements. Due to the conservative nature of the design process with respect to member and connection sizing, the equipment would be designed to resist a *minimum* of 0.4g, and since the *minimum* factor of safety expected from a fastener is 3.0, the equipment would reasonably be expected to withstand the earthquake. Clearly, more study is needed prior to adopting these simple formulae, but a methodology along these lines appears to simplify the design procedure immensely without being significantly underconservative or overconservative.

Maximum Inelastic Response Displacement

Major changes were also made in converting the 1994 UBC drift provisions to the 1997 UBC. First, the lateral displacement or drift calculated from the prescribed design forces was given the symbol Δ_S (previous codes referred to this in words, such as the calculated drift). Next, a symbol Δ_M was established that represented the maximum inelastic response displacement. The relationship between Δ_M and Δ_S is expressed by the equation $\Delta_M = 0.7\ R\ \Delta_S$. The 0.7R factor parallels the use of the 3 $R_w/8$ factor used in the 1994 UBC but differs significantly in magnitude. In addition, drift limitations were set at 2.5 percent of the story height for structures having a fundamental period less than 0.7 second and at 2.0 percent of the story height for longer periods. For the 1994 UBC, these limits had been 1.5 percent and 1.13 percent, respectively. These changes in drift provisions lead to some interesting conclusions.

In the 1997 UBC, the forces used to calculate Δ_S are a function of the term 1/R (e.g., $V = C_vIW/RT$). The equation $\Delta_M = 0.7R\Delta_S$ cancels out the R factor so that Δ_M is a function of 0.7 times the unreduced (i.e., R = 1.0) earthquake forces. In comparison, in the 1994 UBC, the forces used to calculate the equivalent to Δ_S were a function of $1/R_w$ (e.g., $V = ZICW/R_w$). Thus, the equation $\Delta_M = (3\ R_w/8)\Delta_S$ canceled out the R_w factor so that Δ_M was 3/8 of the unreduced earthquake forces. Assuming the unreduced earthquake forces are the same in both codes, the change in defining the maximum inelastic response displacement results in design drifts nearly twice the values prescribed in the 1994 UBC (i.e., 0.7 divided by 3/8 = 1.87). Because the drift limits were also increased (i.e., from 1.13 percent and 1.5 percent to 2.0 percent and 2.5 percent, respectively), the increased design drifts, in themselves, do not significantly affect lateral force structural design processes, since both sides of the equation increase similarly. However, this large increase in design drift has a tremendous effect on the design of structural elements not part of the lateral force resisting system, the design of nonstructural elements sensitive to drift such as exterior wall panels and window systems, and the design of seismic joints and building separations. It is also important to note that since the code earthquake forces can be much larger in the 1997 UBC than the 1994 UBC due to near-fault effects, generally increased soil factors, and other limitations, buildings designed under the 1997 UBC are more likely to be affected by the maximum interstory limits, resulting in significantly stiffer buildings (Freeman 2000).

A recent study of the 1997 UBC drift provisions on 16 buildings indicates that there are technical bases for the 2.0 and 2.5 percent drift limits and the $\Delta_M = 0.7R\Delta_S$ equation, but other limitations and poorly defined procedures can lead to unreasonable requirements for non-structural elements governed by drift and for deformation compatibility. For most of the buildings studied, the calculated drifts will not exceed and can be significantly less than the 1.13 to 1.5 percent drift limits specified in the 1994 UBC if the designer is allowed to use common sense and good engineering techniques. However, since the drift provisions in the 1997 UBC are significantly more complicated than previous editions and have additional restrictions, for the designer who does not take the trouble to "read the fine print" or who does not have unlimited budget to run multiple analysis iterations, the 1997 UBC drift provisions can lead to unreasonable consequences for the design of drift-sensitive elements, and use of these complex drift provisions can lead to unnecessary redesign of perfectly good designs (Freeman 2000).

Furthermore, in Section 1630.10.3 of the 1997 UBC, for calculation of drift, it is not immediately apparent why Equation 30-6 may be disregarded while Equation 30-7 may not, since both equations tended to give very similar minimum base shears for the studied buildings. In previous versions of the UBC, the minimum design base shear was determined by only one equation that could be disregarded during determination of drift. Recently, debate in SEAOC has focused on whether the inclusion of Equation 30-7 in UBC drift computations was a procedural error or whether the omission of the exception was deliberate. Nonetheless, in the 1997 UBC, the use of equation 30-7 as well as minimum force limitations, absence of a maximum constant displacement cut-off, and limitations on mathematical modeling can result in unreasonably large estimates of drift (Freeman 2000).

Use of the Maximum Considered Earthquake

For the 2000 IBC (1997 NEHRP), the basis for the design response spectra became 2/3 of the Maximum Considered Earthquake (MCE) instead of the 475-year average return period generally accepted as the basis for all previous editions of UBC and 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 data base. A Probabilistic Seismic Hazard Analysis (PSHA) provides a relationship between PGA and average return period, P_r. Examples of the significant changes that resulted from switching from the 475-year average return period to the 2475-year average return period include Charleston, South Carolina, where the design forces were approximately doubled, and Boise, Idaho, where the design forces were reduced by approximately 50%. Figure 2 shows a graph comparing a typical California Zone 4 city with sites in Seattle, Washington; Salt Lake City, Utah; New Madrid, Missouri; and Charleston, South Carolina. In the previous editions of both UBC and NEHRP, the design earthquake was based on the 475-year earthquake. Changing the basis from the 475-year earthquake to a 2475-year earthquake radically altered the seismic design for numerous cities across the U.S., including the above cities.

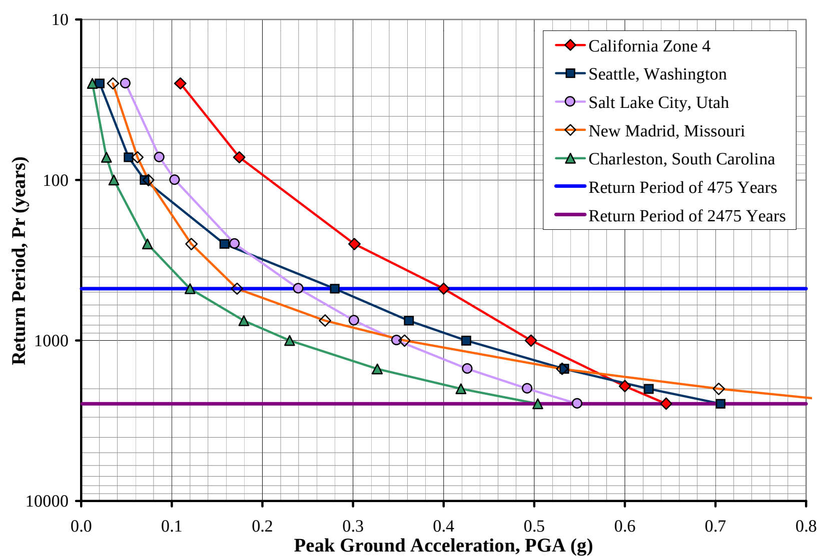

Figure 2. Peak Ground Accelerations for sites in various cities.

By selecting a significantly longer return period for design, the seismic codes have also significantly increased the differences in design philosophy between seismic and non-seismic design criteria. Prior to the 2000 IBC, the average building lifespan was assumed to be approximately 50 years, and design loads were generally scaled to match this assumption. Wind loads as well as snow loads are based on a 50-year return period, and flood hazard maps are generally based on 100-year and 500-year return periods. In the authors' opinion, the selection of a 2475-year design earthquake may seem extreme in light of the much shorter design life-span of a typical building. For two sites that have similar PGAs for the 2475-year return, one may be subjected to large PGAs for earthquakes with a 475-year return and the other site may be subjected to relatively minor PGAs for earthquakes with a 475-year return. Yet both sites would be rated equal according to the new criteria, whereas the latter case actually has significantly less risk of damage during a building's normal life-span than the former case.

Further evidence that the switch to the 2475-year design earthquake may be inappropriate is presented in Table 1. A study by the Federal Emergency Management Agency (FEMA 2001) estimated the future annual losses due to earthquakes in U.S. metropolitan areas. Since the number of buildings and the size of the population living in a given area would appear to be reasonably correlated, we calculated an *approximate* annual per capita loss for each metropolitan area. Looked at another way, the data can be used to approximate a normalized or relative seismic risk associated with each area.

Table 1. Approximate per capita estimated annual loss for selected U.S. metropolitan areas.

Metropolitan Area	**Estimated Annual Loss**	**Metropolitan Population**	**Per Capita Estimated Annual Loss**
San Francisco, CA	$346,000,000	1,731,183	200
Oakland, CA	$348,000,000	2,392,557	145
San Jose, CA	$242,000,000	1,682,585	144
Ventura, CA	$89,000,000	753,197	118
Los Angeles, CA	$1,100,000,000	9,519,338	116
Santa Rosa, CA	$51,000,000	458,614	111
Riverside, CA	$356,000,000	3,254,821	109
Anchorage, AK	$25,000,000	260,283	96
Santa Barbara, CA	$33,080,000	399,347	83
Orange, CA	$214,000,000	2,846,289	75
Reno, NV	$18,000,000	339,486	53
Seattle, WA	$128,000,000	2,414,616	53
Portland, OR	$98,000,000	1,918,009	51
San Diego, CA	$127,000,000	2,813,833	45
Tacoma, WA	$28,300,000	700,820	40
Salt Lake City, UT	$39,000,000	1,333,914	29
Charleston, SC	$13,300,000	549,033	24
Sacramento, CA	$39,000,000	1,628,197	24
Las Vegas, NV	$28,000,000	1,563,282	18
Memphis, TN	$17,000,000	1,135,614	15
St. Louis, MO	$34,000,000	2,603,607	13
Boston, MA	$23,000,000	3,406,829	7
New York, NY	$56,000,000	9,314,235	6

Note that this table is based on limited data and should only be viewed as an approximate measure of relative risk. However, the table shows that California cities are the most prone to significant damage on an annual per capita basis. Consequently, it appears that the seismic zones from the 1994 UBC may have been more appropriate (i.e. San Francisco and Los Angeles – Zone 4; Portland, Seattle, and Salt Lake City – Zone 3; Charleston and Boston – Zone 2A; etc). The result of these modifications are clear; a number of cities will now have seismic design criteria significantly more stringent than have ever been required, and in some cases, significantly more stringent than many areas of high seismicity in California. The authors believe that in many cases, this jump in design criteria appears unwarranted given the extremely low probability of the 2475-year earthquake occurring within the lifetime of a given structure. Another unintended potential consequence of the change in seismic design levels is that much of the existing building stock in an number of affected cities is likely to be unjustifiably deemed inadequate, thus creating an economic burden relating to expensive seismic upgrades that provide only small improvements in real performance during the life of the structure. The true cost of this change may not be realized for many years.

Redundancy/Reliability Factor

In the years since the development of modern seismic building codes (circa the 1973 UBC), it has been perceived that as detailing requirements have become more stringent, seismic engineers have been gradually reducing the relative number of lateral force resisting elements in structures. The rho factor was developed to help “encourage” the engineer to increase the number of lateral force resisting elements to a “reasonable” level (SEAOC 1999). The rho factor depends solely on the floor area of the structure and the maximum ratio of element shear to story shear. The rho factor formulation does not take into account nonlinear behavior (i.e. ductility and overstrength) of individual elements or the structure as a whole, or the ability of the structure to redistribute loads, all of which are arguably necessary to determine redundancy and reliability. Consequently, but not unpredictably, a significant number of unintended and undesirable consequences have occurred as a result of the addition of the rho factor. Specific documentation of these unintended consequences is presented elsewhere (Searer 2002).

A study of the rho factor by the Structural Engineers Association of Northern California recently concluded that the current formulation of rho is “deeply flawed” (SEAONC 2001). Since the unintended consequences of rho are significant and can have serious and potentially adverse effects on structural design, the authors recommend that the rho factor be removed from the code until such time as the rho factor is modified to produce consistent and accurate assessments of redundancy and reliability.

Recent Developments and Conclusions

Considering the large number of recent changes to the building code, the next logical step is to question whether these changes are beneficial or detrimental to the overall practice of structural engineering. A recent study conducted by the Design Practices Committee of the Structural Engineering Institute Business and Professional Activities Division (SEI-BPAD 2001) tested the ability of engineers to follow current design provisions. In an eye-opening study, 22 volunteer engineers with an average experience of more than 12 years were given the task of designing a cantilevered concrete shear wall for seismic loads. Calculated values for the design shear varied by more than 200%, horizontal shear reinforcing varied by more than 800%, and longitudinal boundary reinforcing varied by more than 900%. The study concluded that in general, even experienced practicing engineers are unwilling or unable to follow the complexities in the current building code. In a recent survey by the Structural Engineers Association of California (SEAOC 2001), practicing engineers most often identified the need to simplify the building code as the most pressing issue facing the structural engineering profession. Clearly, when engineers themselves admit that the codes are too complicated and too difficult to use, it is time for the code-writing organizations to take heed.

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