Ninth Canadian Conference on Earthquake Engineering
Ottawa, Ontario, Canada
26-29 June 2007

ARE WE REALLY LEARNING FROM EARTHQUAKES? DECLINING QUALITY AND INCREASING COMPLEXITY OF MODERN BUILDING CODES

G.R. Searer[1], T.F. Paret[2], S.A. Freeman[3] and J. Valancius[4]

ABSTRACT

Today's codes and standards are substantially more complex than those of previous generations. Although these changes are typically introduced under the universal mantle of "lessons learned" from prior earthquakes, many of the changes have nothing to do with "lessons learned". Codes should be based on the fundamentals of mechanics and engineering and not on a patchwork of requirements and equations that are supposed to correct so-called "lessons learned" from prior earthquakes. Examples are presented that show the complexity of codes, particularly provisions dealing with the lateral force resisting systems of structures. Suggestions for improving these codes are presented.

Introduction

In general, today's codes and standards are substantially more complex than those of previous generations -- even restricting one's focus to 'modern' codes. Seismic provisions have ballooned from 14 half-size pages in the 1976 Uniform Building Code (UBC) to 47 full-size pages in the 2003 International Building Code (IBC), and the seismic provisions for each structural material have ballooned even more. Although these changes are typically introduced under the universal mantle of "lessons learned" from prior earthquakes, many of the changes have nothing to do with "lessons learned". This paper addresses whether or not the increasing complexity of today's codes constitutes a net improvement or net decline in the state of engineering.

The increasing complexity of current codes has resulted in increasingly garbled provisions that adversely affect ability of engineers to properly design the critical elements of structures and thus the level of the safety that these designs provide -- the very thing that the codes ought to ensure. This complexity adversely affects the standard of care in engineering, and makes proper review of a design very difficult for building officials, plan checkers, and peer reviewers. In some cases, the provisions have become so garbled and convoluted that it is difficult to understand the reasoning behind the changes or even what the code provisions mean. Indeed, in a relatively recent survey conducted by SEAOC, the most common request of engineers was for building codes to be made more simple and easier to understand (SEAOC 2001).

Codes should be based on the fundamentals of mechanics and engineering and not on a creative

[1]Consultant, Wiss, Janney, Elstner Associates, Inc., 2550 N. Hollywood Way, Suite 500, Burbank, CA 91505
[2]Senior Consultant, Wiss, Janney, Elstner Associates, Inc., 2200 Powell Street, Suite 925, Emeryville, CA 94608
[3]Principal, Wiss, Janney, Elstner Associates, Inc., 2200 Powell Street, Suite 925, Emeryville, CA 94608
[4]Consultant, Wiss, Janney, Elstner Associates, Inc., 2550 N. Hollywood Way, Suite 500, Burbank, CA 91505

patchwork of requirements and equations invented to correct so-called “lessons learned” from prior earthquakes. In the development of earlier codes, complicated issues were discussed, and it was sometimes concluded that there was no easy solution to resolve the problem; the code would state that the engineer should address the issue using judgment and experience. This approach has been abandoned because it necessarily leaves responsibility to the engineer of record; today, rigorous codification of complex and difficult-to-define issues is commonplace. Examples are presented that show the complexity of codes, particularly provisions dealing with the lateral force resisting systems of structures. Suggestions for improving these codes are provided.

Vertical Earthquake Component

During the 1994 Northridge earthquake, some of the peak vertical accelerations recorded were substantially larger than the two-thirds of the peak horizontal accelerations usually assumed by engineers. Consequently, some engineers initially postulated that the relatively large vertical accelerations measured may have been a major or primary cause of damage during the Northridge earthquake (EERI 1997). Actually, as it turns out, the Northridge earthquake did conform to the traditionally accepted ‘two-thirds’ relationship (Bozorgnia *et al*. 1995) and the idea that vertical accelerations played a significant role in damage from the Northridge earthquake has been disproved. Specifically:

- In a study of 36 parking garages (Palaskas *et al.* 1996), vertical accelerations were not found to have caused damage during the Northridge earthquake; the authors of the study found that unless the peak vertical ground accelerations were greater than about 0.3g, factored design loads that did not include a vertical earthquake component controlled the design. Furthermore, it appears as though this study used an artificially low design live load (only 16 psf, rather than the 30 psf to 50 psf mandated by current code), and appears not to have reduced the effects of vertical accelerations to account for slab mass participations of less than 100%; consequently, it appears that the peak vertical accelerations must be even higher before they will begin to control the design -- and even then are not likely to cause collapse (due to significant expected overstrength, inherent redundancy, damping, etc.).

- A study by the Federal Highway Administration concluded that high vertical accelerations during the Northridge earthquake were not identified as the primary cause of damage in any of the 2000 bridges in the earthquake’s epicentral region (Cooper *et al.* 1994).

- After extensive review of data from the Northridge earthquake, the consensus of the Structural Engineers Association of California (SEAOC) was that explicit consideration of vertical ground motion was simply not justified (SEAOC 1996).

- More recently, the Los Angeles Tall Buildings Structural Design Council concluded that tall buildings are generally not susceptible to vertical accelerations and that only if the structure has long cantilevers or construction that may be sensitive to vertical accelerations, should vertical accelerations be considered in the design (LATBSDC 2005).

Despite all evidence to the contrary, we note with some dismay that even very recently, the disproved theory that vertical accelerations caused significant damage is still perpetuated by a number of engineers. More importantly, soon after the Northridge earthquake, the building code was modified to include a vertical earthquake component -- without any substantive evidence to justify the change.

Why the Vertical Earthquake Component Was Really Added to the Code

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 reduced live (L) and earthquake lateral forces (E). However, more recent strength design requirements in *ACI-318* and the *Manual of Steel Construction - LRFD*, for example, (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.

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

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 axial load 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. 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 the 1.4 and 1.2 load factors in the ASD and strength design provisions, respectively (SEAOC 1999; Bachman and Bonneville 2000).

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

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 the equivalent of approximately 20% more dead load, effectively 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 by substituting in the vertical and horizontal components as defined in Equation 3:

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

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 caused the vertical component to become subject to the "±" sign in the load combination formula. Consequently, once the vertical earthquake component was combined with the horizontal earthquake component, the attempt to force parity with prior codes was unintentionally altered.

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% equivalent dead load, thus reducing the total sustained downward 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, making an already cumbersome process even less manageable. More importantly, some 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 -- suddenly appeared to be unstable (Zsutty 2000). This unforeseen and unintended problem is exacerbated by the strength design lateral 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, making much of the existing building inventory appear to be unstable and 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 -- ostensibly to better represent structural behavior and improve design efficiency -- 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 as much as 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, not the least of which is -- as one of the authors is fond of saying -- that there is no evidence that suggests that anyone, anywhere, at any time has ever gotten his or her big toe stuck under an element that has experienced transient uplift during an earthquake.

1. Buildings typically fail by falling downwards, not upwards. While a structure might experience transient vertical accelerations that reduce the net downward resultant loads, the overall upward accelerations typically do not overcome gravity even for an instant, and within a very brief time, the vertical accelerations have reversed and add to the overturning resistance.

2. Neglecting dead loads that are actually present results in increased tension demands (and capacities) in foundation elements, and necessarily results in corresponding increased compression loads on other elements. Allowing uplift limits the amount of compression loads in certain elements, and precluding or delaying uplift has the potential to create buckling and compression failures where none would have occurred.

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

4. Even if overturning of an individual element overcomes the dead load tributary to it, if the deformations are significant enough in most structures, additional dead load will be mobilized through deformation and/or catenary action of members that frame into the uplifting element.

5. Finally, rocking is generally considered to be beneficial to the overall response of buildings to earthquakes (provided other elements are ductilely detailed) and tends to increase effective damping and decrease overall building response.

Recent Developments Regarding the Vertical Earthquake Component

In an apparent effort to make allowable stress design for overturning and uplift equivalently overconservative and inefficient as strength design, *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-05) 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 authors believe 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 Regarding the Vertical Earthquake Component

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, with the observation that the increased number of load combinations remains burdensome. However, in the authors' 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.

$$(1.2)\ D \pm (1.0)\ E_h + (f_1)\ L + (f_2)\ S \tag{11}$$

and

$$0.9D \pm 1.0\ E_h \tag{12}$$

Vertical Distribution of Seismic Forces

With the development of ASCE 7-05, vertical distribution of seismic forces has been unnecessarily complicated. Older codes, including the 1997 UBC, required that for short period structures, seismic forces be distributed over the height of the building, weighted by the relative height, h_x, and mass, w_x, of each story -- to simulate the inertial effects of the first mode, which can be linear, concave, or convex -- depending on the structural system used -- but which can be reasonably approximated by a linear relationship. For longer period structures, a concentrated force or "whiplash" force, F_t, was added at the top of the structure to simulate the effects of higher modes, and the remainder of the base shear, V, was distributed based on the mass and height of each floor (Equation 13).

$$F_x = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i} \tag{13}$$

The amount of the second mode design force varied linearly with the period of the structure and was never more than 25% of the total base shear.

In ASCE 7-05, forces are distributed over the height of the structure by a fictitious parabolic weighting scheme, where k varies from 1 to 2, depending on the period of the structure (Equation 14).

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{14}$$

The parabolic weighting promulgated by ASCE 7-05 does nothing to further the understanding of how earthquakes affect structures. Only relatively long period structures might benefit from a better approximation of how their numerous modes interact; however, we note that long-period structures are already required to have a dynamic analysis, making Equation 14 moot. According to the NEHRP commentary (2003), this change was made because the fundamental vibration mode lies "approximately between a straight line and a parabola…" but no backup for this claim is provided, and it is not clear how this added complexity will significantly improve the performance of any buildings.

A Plethora of Patches

A perusal of the building codes shows numerous examples where illogical changes or patches have been made to the code over time.

Out-of-Plane Anchorages of Walls

Consider the design of out-of-plane anchorages of walls supported by flexible diaphragms. The 1997 UBC requires that “the strength design forces for steel elements of the wall anchorage system shall be 1.4 times the forces otherwise required…” Similarly, “the strength design forces for wood elements of the wall anchorage system shall be 0.85 times the force otherwise required…”

Not only are these requirements poorly worded (leaving the incorrect impression that the requirements do not apply to allowable stress design), but they also provide confusing instructions to the designer (e.g. For a steel anchorage that is fastened to a wood member on one end and the concrete wall on the other, what design force should be used?) and increase the likelihood of a mistake in the design process. The commentary is completely silent on why these changes were made or what the intent was. Furthermore, the small increases or decreases in design load (particularly the 0.85 factor for wood) will have little if any effect on the overall behavior of the connection, given the fact that both the design forces and the design strengths are at best crude approximations of actual behavior. The 1.4 load factor for the steel may end up forcing the ultimate failure into the concrete, particularly where drilled-in chemical anchors are used during upgrade of existing tilt-ups to current code. A more rational design procedure than the one currently specified in the UBC or the IBC has recently been suggested (Freeman *et al.* 2002) but in-depth discussion of the procedure is beyond the scope of this paper.

Design of Flexible Diaphragms Supporting Rigid Walls

Another good example of a poorly worded and ill-considered patch to the code is the requirement in UBC Section 1633.2.9 that design forces for flexible diaphragms that support walls or frames of masonry or concrete shall be determined using an R not exceeding 4. Based on the Blue Book Commentary, this change was added to react to the possibility that reinforced masonry frame elements might be used to provide lateral support for wood diaphragms. Since it was feared that the inelastic behavior would be forced into the presumed nonductile wood diaphragms by the assumed ductile masonry frames, a patch was added to preclude the diaphragms from being designed with an R greater than 4 (SEAOC 1999). Note that while the patch was purportedly intended to address masonry frames, it had the unintended consequence of affecting the design of most if not all rigid wall, flexible diaphragm structures. Since typical tilt-ups have concrete wall panels and a wood diaphragm, they are designed using an R of 4.5; however, the patch in Section 1633.2.9 requires that the diaphragm be designed with an R of 4, resulting in a 12.5% increase in diaphragm design forces. Since the increase is triggered by a code section outside the portion where forces are typically computed, it is easy for designers to overlook this provision. Note that the small increase in design force is dwarfed by the uncertainty associated with the approximations in the design procedure and variations in material strengths and construction quality, and that the increase appears unlikely to globally improve the overall behavior of the typical structure.

Strength Design of Concrete and Masonry

Strength design has also been unnecessarily complicated by unwarranted tampering with the code. Section 1612.2.1 of the 1997 UBC presents typical strength design load combinations. A small footnote to the table, under the label “Exceptions” states “Factored load combinations of this section multiplied by 1.1 for concrete and masonry where load combinations include seismic forces”. The requirement lacks a verb -- which makes the requirement meaningless and nonsensical. Furthermore, if Bachman and Bonneville (2000) are to be believed, the intent of this footnote was to increase seismic design forces by 10% to match ASCE 7 requirements; but this requirement may actually decrease expected performance, since Bachman and Bonneville report that it has the effect of promoting shear-critical behavior in shear walls -- particularly when combined with the unanticipated side effects of the vertical earthquake component. While the authors of this paper have not confirmed Bachman and Bonneville’s claim, if true, the 1.1 factor is not of benefit to the seismic performance of buildings and should be deleted.

Whatever the meaning of the poorly worded footnote exception, in the authors’ opinion, it is unacceptable to tamper with commonly used load combinations via the use of a small footnote that has the effect of increasing the design level forces; footnotes should be used to explain applicability or to reduce design forces, but not to increase them since footnotes are easily overlooked.

Redundancy

No paper regarding problems with and unnecessary complexity in current codes would be complete without at least briefly touching on the redundancy/reliability factor, ρ (rho). 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 (SEAOC 1999). 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. However, when viewed with the clarity of hindsight, the addition of rho would not have solved these problems. Furthermore, the rho factor was found to have negative and unforeseen influences on the design of buildings. More in-depth discussions regarding the rho factor in the 1997 UBC can be found in Searer (2000), SEAONC (2001), Searer and Freeman (2002a), and Searer (2006).

The recently published ASCE 7-05, the basis for the 2006 IBC, now contains another formula for rho (after attempts to tweak rho in other documents failed). 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 authors believe that this formulation is more rational and more defensible than the formulation in the 1997 UBC, we anticipate that there may be difficulties in implementing this requirement. We note 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. While the authors agree with the need for redundancy in design, the resultant chaos of trying to quantify redundancy has significantly complicated and worsened the building code.

Current Trends

The authors believe it important to stress that the current trend of adopting material specifications (such as steel (AISC), concrete and masonry (ACI), wood (AFPA), etc.) by reference and then modifying or "tweaking" the published codes via language in the adopting code is not good practice. Both the IBC and ASCE 7-05 frequently take consensus material specifications and then tweak them, causing confusion in the code adoption cycles and essentially pre-empting the published versions of the specifications. There is a significant risk that designers will use a published material specification without recognizing that it has been tweaked. While designers are required to purchase numerous specifications that formerly were incorporated directly into the code and are then forced to attempt to correlate changes required by the adopting code agency, code development profit centers for each of these materials reap the financial benefits of this flawed system. If AISC, ACI, AFPA, etc. are not willing to share their specifications for the good of the engineering community, and if ASCE 7-05 and the IBC are not willing to purchase the rights to the specifications and include them in the code for the good of the engineering community, then any required changes should be made through the appropriate material specification-writing committees *or not at all*.

Nonstructural Component Design

As a tribute to poorly worded and unjustified modifications to codes, consider the nonstructural component design portions of the 1997 UBC. The 1997 UBC was intended to be a transition from the UBC to the

IBC. The 1994 UBC had a very simple formula for determining anchorage forces, F_p, for nonstructural components (Equation 15), where Z is the seismic zone factor (0.4, 0.3, 0.2, etc.), I_p is the importance factor (either 1 or 1.5), C_p is the component factor (either 0.75 or 2.0), and W_p is the weight of the nonstructural element.

$$F_p = Z I_p C_p W_p \tag{15}$$

Based on a largely faulty assumption that anchorage forces should increase linearly from bottom to top of a building, the 1997 UBC adopted a triangular formulation and broke down the original C_p factor into two unjustifiable terms -- an amplification term, a_p, and a response reduction factor, R_p -- to produce Equation 16.

$$F_p = \frac{a_p C_a I_p}{R_p}(1 + 3\frac{h_x}{h_r})W_p \tag{16}$$

Independent analysis of the same data that was used to justify the changes incorporated into the 1997 UBC shows that the 1994 UBC formula was conservative for nearly all cases, with the sole exception of anchorage forces near the roof (Kehoe 1998 and Searer and Freeman 2002b). Rather than modify the existing formula, which had been used for decades without significant evidence of widespread dysfunction, the formula was made substantially more complicated without justification.

Furthermore, the code was changed from a small relatively straightforward table that provided the C_p factor to a nearly one-and-a-half-page-long table with a_p and R_p terms, and with about as many footnotes as entries in the table; some of the footnotes can as much as double the design forces. Again, the authors believe that the use of footnotes, which can often be overlooked, to increase design level forces should be strongly discouraged. We note that ASCE 7-05 has reduced the factor associated with the hx/hr term from 3 to 2, which will tend to reduce the design forces by 25% -- a good decision, given the overconservative results produced by this formulation. However, we also note that the number of table entries for a_p and R_p in ASCE 7-05 has approximately doubled, with a_p ranges from 1 to 2.5 and R_p ranges anywhere from 1 to 12, giving results that will vary by more than an order of magnitude, depending on what is being designed and what assumptions the designer makes. It is not clear that this depth of complexity is necessary or that it adds to the overall efficiency of the engineer or that it simplifies or reduces the overall cost of construction. Precision does not necessarily equal accuracy or efficiency.

Excessive Precision

In the authors' opinion, seismic codes are becoming excessively precise, with no significant increase in accuracy. In the 1976 Uniform Building Code, there were only six defined types of lateral force resisting systems. In ASCE 7-05's Table 12.2-1, there are now 83 defined types of lateral force resisting systems, each with its own Response Modification Coefficient (R), seismic overstrength factor (Ω_0), Deflection Amplification Factor (C_d) and five different -- and completely arbitrary -- height limitations depending on the Seismic Design Category. In ASCE 7-05, the response modification factors range from 1 to 8; the overstrength factors range from 1 to 3; and the deflection amplification factors range from 1 to 6.5; all of which vary by increments as small as ¼. Further, the load combinations in chapter 12 of ASCE 7-05 multiply the seismic forces by factors with as many as three significant figures. We view these developments with dismay, as should every practicing engineer and every researcher who well knows that the problem of seismic behavior cannot be sensibly parsed so finely.

While a few notable types of lateral force resisting systems have been added (such as precast concrete moment frames, steel plate shear walls, eccentrically braced frames, and buckling restrained braces), one questions whether there really has been an approximately fourteen-fold increase in the number of typical lateral force resisting systems in the past 30 years, or if this portion of the code has become unnecessarily complex, particularly since the response modification factors, overstrength factors, and deflection amplification factors (as well as the height limitations) are essentially made-up, committee-consensus numbers with very little technical basis. Furthermore, given that these factors are essentially made-up, one questions how three significant figures are justified in the load combination sections. The profession would probably be better served by a smaller but more rational table of typical lateral force resisting systems than by a massive, overly complicated table that lacks justification.

Seismic Drift

Given the significant nonlinear behavior inherent in structures during large earthquakes, predicting seismic drift is a complicated matter. Unfortunately, the 1997 Uniform Building Code (and to some degree ASCE 7-05) unnecessarily complicates the prediction of maximum drift by requiring prescribed minimum base shears, requiring prescribed limits on calculated periods, and failing to account for the constant displacement demand region in the demand response spectrum -- in whatever period range it occurs. While a detailed description of the drift provisions is beyond the scope of this paper, additional discussion is presented in Freeman and Searer (2000).

Wind Loads

Similar to seismic loads, wind load provisions have recently undergone a massive and unjustified metastasis. Wind design provisions have exploded from two and a half pages in the 1997 UBC to 60 pages in the 2005 ASCE 7-05 standard. One might expect such an increase in complexity of provisions if there had been widespread unforeseen destruction that demonstrated numerous serious problems with the building code; however, no such catastrophe has occurred or is predicted to occur. Wind-caused failures in general are rare in non-hurricane areas and, at least on the west coast of the United States, are typically the result of poor design, poor construction, or deterioration. In the middle of the United States, tornadoes have historically caused damage to single family residences and mobile homes, but increasing wind design complexity will likely do very little to stop damage to these types of structures from wind-borne objects (e.g. trees, cars, telephone poles, etc.) moving at 150 to 300 miles per hour. Even in US coastal communities along the eastern seaboard and Gulf of Mexico, where increasing design forces might be justified, why not just increase the design forces instead of making wind design extremely complex? The old UBC provisions were straightforward, elegant, and easy to use. In contrast, the ASCE 7-05 provisions are overly and unnecessarily complicated. The authors are not aware of any justification for the huge and onerous increase in the complexity of wind design load provisions.

Conclusions

In each of the above sections, the authors have identified areas of the code that are unnecessarily complex, confusing, or poorly worded, and instead were developed for purposes unknown or were proposed as last-minute patches to problems either real or supposed. A primary concern among engineering practitioners is that building codes are rapidly becoming too complex or confusing -- so much so that some provisions are unworkable or unintelligible. The engineering community needs to rally behind a drive to simplify the building codes, delete portions of the codes that are not justifiable or are overly complex, and rewrite those unclear portions of the codes that are justifiable so that the intent of the provisions is clear. Furthermore, commentaries should describe why each provision of the code exists and its intent, and should include an unbiased review of provisions that are controversial.

References

Adamo & Associates, Inc., 2006. "Images from the Northridge, California Earthquake of January 17, 1994," webpage: http://www.nvo.com/adamoandassoc/northridge11994/index.nhtml, last accessed September 8, 2006.

Bachman, R.E. and Bonneville, D.R., 2000. "The Seismic Provisions of the 1997 Uniform Building Code," *Earthquake Spectra*, Volume 16, Issue 1, Earthquake Engineering Research Institute (EERI).

Bozorgnia, Y., Niazi, M., and Campbell, K.W., 1995. "Characteristics of Free-Field Vertical Ground Motion in the 1994 Northridge Earthquake," *Earthquake Spectra*, Volume 11, Issue 4, EERI.

Cooper, J.D., Friedland, I.M., Buckle, I.G., Nimis, R.B., and Bobb, N.M., 1994. "The Northridge Earthquake: Progress Made, Lessons Learned in Seismic-Resistant Bridge Design," *Public Roads*, Volume 58, No. 1, US. Department of Transportation, Federal Highway Administration, McLean, VA.

Earthquake Engineering Research Institute (EERI), 1997. "The EERI Annotated Slide Collection," Oakland, CA.

Freeman, S.A., and Searer, G.R., 2000. "Impact of the Revised Earthquake Drift Provisions on Design and Construction" 2000 Structural Engineers Association of California Convention.

Freeman, S.A., Searer, G.R., and Gilmartin, U.M., 2002. "Proposed Seismic Design Provisions for Rigid Shear Wall / Flexible Diaphragm Structures," 7th U.S. National Conference on Earthquake Engineering.

Hart, G.C., Ekwueme, C., Jain, A., and King, S.A., 2003. "Structural Engineering Evaluation of Earthquake Damage to 1121 La Cienega Blvd., West Hollywood California", Weidlinger Associates, Inc., Santa Monica, CA.

International Parking Design, 2006. "Impact of the Northridge Earthquake on Design," webpage: http://www.ipd-global.com/northridge.html, last accessed September 11, 2006.

Los Angeles Tall Buildings Structural Design Council (LATBSDC), 2005. An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region.

NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures: Commentary, 2003. Federal Emergency Management Agency, Washington, D.C.

Palaskas, M.N., He, L., Chegini, M., 1996. "Vertical Seismic Forces on Elevated Concrete Slabs," Practice Periodical on Structural Design and Construction, Volume 1, Issue 3, American Society of Civil Engineers, Reston, VA.

Searer, G.R., 2000. "Evaluation of the Reliability/Redundancy Factor," Wiss, Janney, Elstner Associates, Inc., Emeryville, CA.

Searer, G.R. and Freeman, S.A., 2002a. "Impact of the Reliability/Redundancy Factor on Design," 7th U.S. National Conference on Earthquake Engineering.

Searer, G.R. and Freeman, S.A., 2002b. "Unintended Consequences of Code Modification," 7th U.S. National Conference on Earthquake Engineering.

Searer, G.R., 2006. “Poorly Worded, Ill-Conceived, and Unnecessary Code Provisions” by Gary R. Searer, 2006 Annual Meeting of the Los Angeles Tall Buildings Structural Design Council, Alternative Procedures for Design of Tall Buildings, Los Angeles, CA.

Structural Engineers Association of California (SEAOC), 1996. *Recommended Lateral Force Requirements and Commentary, Sixth Edition*, Structural Engineers Association of California.

Structural Engineers Association of California (SEAOC), 1999. *Recommended Lateral Force Requirements and Commentary, Seventh Edition*, Structural Engineers Association of California.

Structural Engineers Association of California (SEAOC), 2001. “Member Satisfaction and Needs Assessment Survey Summary”, *Plan Review*, February/March.

Structural Engineers Association of Northern California (SEAONC), 2001. “Committees on Assignment: Seismology”, *NEWS* LVI (7), pp. 3-10, San Francisco, CA.

Zsutty, T.E., 1999. “Suggested Simplifications for the 1997 UBC Seismic Design Provisions”, Structural Engineers Association of California 1999 Convention, Structural Engineers Association of California.

Zsutty, T.E., 2000. “The Foundation Design Example 29 in the SDM Volume 1”, letter to Martin Johnson, Chair of SEAOC Seismology Committee, January 14, 2000.