Performance-Based Design of RC Buildings for Wind Loads – Overview and Issues

by J.M. Bracci

Synopsis: The paper provides an overview of the current design methodology for wind load based on ASCE-7 (2002) and IBC (2003). In addition, an attempt is made to identify the issues in developing a performance-based design methodology for wind loading, in particular for reinforced concrete frame buildings. Explicit comparisons of wind and earthquake loading on structural systems are made to leverage a discussion for a performance based wind design methodology. It is demonstrated that significant differences in performance based design methodologies will exist due to the nature of the loadings and the different design philosophies. Future research is required on establishing criteria for appropriate performance objectives and performance levels during serviceability and strength loading conditions.

Keywords: buildings; limit states; performance; stiffness; strength; wind

Joseph M. Bracci is a Professor in the Department of Civil Engineering at Texas A&M University, College Station, Texas, 77843-3136 and is Chairman of ACI Committee 375, Performance Based Design of Concrete Buildings for Wind Loads. He received his PhD degree from the State University of New York at Buffalo. His research interests include experimental testing, analytical modeling, and performance-based design of structures.

INTRODUCTION

Performance Based Design (PBD) or Performance Based Engineering is defined in this paper to be a design methodology in which a structural system is designed to satisfy selected performance objectives. These objectives require the building to be designed to satisfy various performance levels (limit states or capacities) during varying intensity load conditions (demands). For example, building codes require that buildings must be designed to have sufficient strength to withstand ultimate (factored) loading and stiffness to limit deformations and lateral drift for functionality during frequent service loading (IBC 2003).

Performance Based Seismic Design (PBSD) has recently flourished in regions of high seismic risk due to a variety of unsatisfactory structural and nonstructural performances during minor, moderate, and severe earthquakes. In PBSD, performance objectives have traditionally been qualitative in nature such as to ensure equipment operability and immediate occupancy following an earthquake event with a frequent (higher) probability of occurrence, life safety during a design basis earthquake with a moderate probability of occurrence, and that the building will not collapse during a rare earthquake event (FEMA 356, 2000). In earthquake engineering, the general consensus is that inter-story deformations can be directly correlated to seismic damage due to significant nonlinear behavior, rather than forces as in typical Strength Design methods. As an example using quantitative structural response values for reinforced concrete (RC) special moment frame buildings, buildings might be designed to limit inter-story deformations during varying earthquake design events described as follows: 1%, 2%, and 4% of the story height (typically associated with immediate occupancy, life safety and collapse prevention limit states, respectively) during earthquakes with exceedance probabilities of 50%, 10%, and 2% in 50 years [73-, 475-, and 2,475-year average return periods], respectively. It is important to emphasize that these limiting displacement values are based on deterministic system-wide response. In addition, performance levels have also been expressed deterministically using member response limits (such as rotations and curvatures) (FEMA 356, 2000). However, there is a need for better definitions of target performance for structural and non-structural components.

PBSD has also been a major research thrust activity of the Pacific Earthquake Engineering Research (PEER) Center, sponsored by the National Science Foundation. Their research is focusing on the development of a second-generation approach for performance based design for earthquakes (PEER, 2004). The enhanced methodology uses explicate calculations for system performance with direct interest to stakeholders, such as dollars, deaths and downtime, and rigorous probabilistic calculations of system performance based on uncertainties in the earthquake demands, system response, physical damage, and economic and human loss.

This paper provides an overview of the current design methodology for wind loading and attempts to address the issues in defining a performance based design methodology for wind loading, in particular for reinforced concrete buildings. In addition, the paper uses explicit comparison between wind and earthquake engineering to leverage the discussion for performance based wind design (PBWD).

CURRENT BUILDING CODE REQUIREMENTS

Reinforced concrete buildings have traditionally been designed based on the Ultimate Strength Method (ACI 318-02). The performance objective for this method requires that the building be designed to support safely the factored loading according the following general equation:

$$\phi\, U \geq \alpha_i\, L_i \tag{1}$$

where ϕ is the strength reduction factor to account for uncertainty in the nominal capacity;
U is the nominal capacity of the member, such as flexure or shear strength;
α_i is the load factor for loading i;
L_i is the demand created by loading i.

This equation, also called the safety checking equation, represents a member limit state or failure mode. Limit states include flexure, shear, axial, anchorage, etc. The left side of the equation is referred to as the design strength while the right side is the factored load or load combination. ASCE-7 load combinations have been determined according to Turkstra's rule (Turkstra and Madsen, 1980) which states that the maximum combined load usually occurs when one action achieves its maximum while other loads are at average values. In effect, the factored load represents a conservative estimate of the highest load a member will experience during its design life. There is a low probability that the factored load will be exceeded during the member's design life, except under the extreme loads of an earthquake, wind, or progressive collapse caused by a failed member.

The predominant building code load combinations (IBC 2003 and ASCE-7 2002) that consider the effects of wind loading are:

$$1.2\,D + 0.5\,L \pm 1.6\,W \tag{2}$$

$$0.9\,D + 1.6\,W \tag{3}$$

where D is the demand due to the self weight and superimposed dead weight;
L is the demand due to the live load (note that the 0.5 load factor is used for most structures and 1.0 should be used for public assemblies);

W is the demand due to the 50-year return period wind load based on the 3-second peak gust wind speed.

In the current versions of these codes, the product of 1.6 W corresponds to an ultimate wind loading from a probabilistic intensity with a return period of between 500 and 700 years. For buildings with occupancy category III or IV, with an importance factor of 1.15, the product of 1.6 W corresponds to an ultimate loading from a probabilistic intensity with a return period between 1000 and 1400 years. ASCE-7 also provides conversions factors for other mean recurrence intervals that are primarily used for serviceability considerations. One particular event that is often considered in the design process is the wind loading from a 10-year return period, which is determined by multiplying the 50-year mapped wind speed in non-hurricane areas by 0.84 or in hurricane areas by 0.74. But, such factors are based on quasi-static response to wind excitation and are accurate for rigid low-rise buildings. Therefore, the current version of ASCE-7 provides a way for structural engineers to determine the magnitude of the wind loading for a variety of probabilistic events, which can be used to define the structural demand in a PBD methodology, at least for low-rise structures. However, practicing engineers currently need to use their judgment in determining the load level and acceptance criteria for other performance objectives, ie. limiting the inter-story drifts during a 10-year or 50-year wind event to less than about 0.5 in. for cladding damage protection.

Eqs. (2) and (3) require members to have adequate strength to resist the largest anticipated loads that might occur during the design life of the structure. This will be termed the Strength (or Basic Safety) Performance Level in this paper. Thus, the Performance Objective in current codes is that the Strength Performance Level is satisfied during the design basis wind event. Life safety of the occupants should be assured during this loading. ASCE-7 also states that buildings are likely to have capacity to resist higher wind loads due to resistance factors of the materials, conservative design procedures, structural redundancy, and lack of a precise definition of “failure”. In addition, due to these same conservative design principles, structures exposed to the design basis event are expected to merely reach the first significant yield point (FY). As a result, some stiffness reduction may result in reinforced concrete structures due to cracking.

In addition to satisfying Eqs. (2) and (3), another performance objective that is required in building codes is to design the building to have adequate stiffness to limit deflections and lateral drifts during frequent loading, which is typically classified as service or unfactored loading (Serviceability Performance Level). The Serviceability Performance Level for wind loading is typically associated with the perception to motion by people, the vibration sensitivity of working equipment, and the expansion gap between the structural and nonstructural systems. Note that in a structural engineering context, the perception to motion by people and vibration sensitivity of working equipment are related to the dynamic vibration response of the building during live and wind loading. The third item is related to limiting the inter-story displacements during lateral loading to limit damage to the nonstructural cladding. For example, most reinforced concrete frame buildings constructed in low-to-moderate seismic zones have lateral isolation between the structural frame and nonstructural cladding systems on the order of about 12 mm (0.5 in.). Since cladding is very expensive, most design firms design the structure to limit the sway of the structural frame during frequent lateral loading, such as 10-year wind event, be limited to this gap or about 0.25 percent of the story height. However, it should be emphasized that serviceability limit states are not specified in model building codes because they are often subjective and difficult to define or quantify. Therefore, structural engineers are required to use their best judgment primarily based on past experiences to reduce the likelihood of nonstructural damage during frequent wind events.

In addition to strength and serviceability design criteria, ACI 318 also requires in section 10.13.6 that the strength and stability of a structure under factored gravity loading be considered by limiting the ratio of the second order deflections to the first order deflections to 2.5, which corresponds to a stability index of 0.6 (Stability Performance Level). This is typically accomplished by applying a fictitious lateral load to the structural model with factored gravity loading and comparing the results from separate first-order and second-order analyses. Although this performance level has nothing to do with wind loading, it is listed here as many of the parameters that affect the Strength Performance Level also affect the Stability Performance Level.

PERFORMANCE LEVELS

Performance levels or limit states for both structural and nonstructural systems are defined as the point in which the system is no longer capable of satisfying a desired function. In particular, building codes using the Ultimate Strength Method require structural engineers to: (1) provide sufficient ultimate or plastic strength in the building elements or members to resist design basis loading (factored loading) to protect the life safety of the occupants; (2) provide sufficient building stiffness for functionality during service level loading. However for wind loading, currently building codes give no explicit guidance on establishing performance limits and demand loading criteria; and (3) design the building for stability during sustained loading. Note that these performance levels are typically defined deterministically.

Another approach for defining structural performance levels might be based on quantitative procedures using nonlinear pushover techniques (Wen et al., 2004). These quantitative performance levels can be utilized by the designer to supplement the qualitative performance levels in current building codes. Example performance levels that can be identified analytically using nonlinear pushover procedures are:

(1) *First Yield (FY)* – Point at which a member of a story initiates flexural yielding under imposed lateral loading;

(2) *Plastic Mechanism Initiation (PMI)* – Point at which a plastic story mechanism initiates under imposed lateral loading; and

(3) *Strength Degradation (SD)* – Point at which significant strength reduction occurs either due to material non-linearities, geometric non-linearities due to P-delta effect, or due to sudden loss of load carrying capacity triggered by brittle behavior.

For example, consider the portal frame in Fig. 1. Under imposed lateral loading, the story shear force versus inter-story drift can be calculated using pushover techniques. The FY performance level corresponds to an inter-story drift at first member section yielding, shown at the base of the columns. The PMI performance level subsequently occurs after both ends of the beam yield. The sequence and pattern of plastic hinge locations prior to the mechanism formation are important. Both may significantly affect the structural deformability (capacity) in building structures. The SD performance level is defined as the deformation when the strength is reduced by 20 percent of the maximum attained strength. This performance level is highly influenced by the post-yield stiffness of the member sections, which is related to strain hardening of the reinforcement and the second order moments created by P-delta effect. Generally speaking, the SD performance level is difficult to quantify because of the complex modeling requirements. Since structure demands during wind loads are primarily due to unidirectional forces, other important structural characteristics are the overstrength capability of the structure from FY to PMI and from PMI to peak strength. It should be emphasized that structures designed according to current building codes would not have any members subject to significant yielding during a strength level demand event or factored loading combination. Therefore, current building codes restrict building performance to be within/near the FY performance level. However in overload scenarios beyond the code requirements, the PMI and SD performance levels could be considered.

A key input parameter required in identifying such quantitative performance levels is the imposed vertical distribution of lateral loading or deformations. However for regular low- to mid-rise buildings that are considered rigid for wind purposes (fundamental periods less than 1 second), the imposed lateral forces used in static analyses should be consistent with those in model building codes.

PBD METHODOLOGY FOR WIND LOADS - ISSUES

Performance based design methodologies for earthquake loading were summarized previously. However, performance based design methodologies for wind loading may be considerably different than those used for earthquake loading due to the different: (1) design philosophies for wind and earthquake loading, ie. anticipated elastic vs. inelastic behavior for wind and earthquake loadings, respectively; (2) participation of nonstructural elements; and (3) load effects on the building. Below each of the issues are addressed.

Design Philosophy

One factor that may lead to different performance based design criteria used for wind and earthquake engineering is the different design philosophies for each loading. Earthquakes tend to excite structural systems by exposing them to severe cyclic loading that is frequency dependent. The best representation to emphasize the effect of earthquakes on structural systems is by using an elastic response spectrum, which defines that peak spectral acceleration of single degree of freedom systems with varying fundamental periods for a constant level of equivalent viscous damping. Fig. 2 shows an example time history and corresponding elastic response spectrum with 5 percent damping for the 1940 El Centro earthquake in California (0.34 g peak ground acceleration). From the time history plot, cyclic loading is evident and is highly variable. The response spectrum for this particular earthquake shows that significant amplification occurs when structural periods are less than 1.0 seconds. For higher natural periods, the magnitude of the peak accelerations is much smaller, which implies that buildings with longer fundamental periods may be less vulnerable to forces generated by earthquakes. At the same time, however, they may be prone to larger deformations during earthquakes which may amplify second order effects due to P-delta.

In addition to earthquake loading being cyclic and frequency dependent, the intensity of the design seismic event in high-risk earthquake zones is often very large, making design based on elastic response economically impractical. Therefore, structural systems are intentionally designed to sustain structural damage in an effort to resist earthquake forces by hysteretic energy dissipation of the structural members during cyclic response. By doing so, design lateral forces can be significantly less than those expected during elastic behavior. However since inelastic response is expected, critical member sections must have appropriate detailing to ensure significant inelastic deformability (or ductility). Since code-based design acceleration spectra typically decreases with increasing fundamental periods, structural damage tends to increase the fundamental period of the structure and limit the maximum accelerations (or forces) that the building will experience. So during a code-based design earthquake (which has historically been the 10% probability of exceedence in 50 years [about 500 years return period] or more recently taken as two-thirds of the 2% probability of exceedence in 50 years, which corresponds to about 2500 years return period), significant structural damage is expected. However, the preservation of life safety can be accomplished due to stringent reinforcement detailing required to tolerate significant deformation prior to failure.

In contrast, buildings designed for wind loading are expected to respond primarily in the elastic region or at the point of incipient yielding during a probabilistic event that is similar to that used for earthquake design. The reason for this is that the nature of the applied loading on low- to mid-rise structures (regular structures that are assumed to be rigid laterally or not prone to dynamic amplifications from wind loading) due to wind is, for the most part, considered to be statically applied only in one direction and not dependent on frequency. For more flexible structures (fundamental building periods greater than about 1 second), these assumptions are not valid and building flexibility (dynamic amplification) must be considered in both the analytical procedure and wind tunnel testing. For regular structures, ASCE-7 in the simplified and analytical procedures utilizes expressions for wind pressures that are applied statically. Using the Ultimate Strength Method, member sections are designed for sufficient strength to resist ultimate factored loading. Since Eqs. (2) and (3) do not consider the effects of material overstrength (which can be on the order of 10-20 percent increase for reinforced concrete) and conservative design principles using load factors and strength reduction factors, then the building response during a design basis wind event should be primarily in the elastic range, possibly with reduced stiffness due to cracking and without significant yielding.

Fig. 3 highlights the expected force vs. deformation behavior for both earthquake and wind loading. The design basis earthquake force is significantly less than the anticipated elastic forces, but significant inelastic deformations are expected. However, structural behavior during the design basis wind load is either elastic or on the verge of incipient yielding due to material and system overstrengths, and conservative design principles. Therefore, these structures have an inherent, but limited, system overstrength. However, overloading beyond the peak strength resistance may lead to uncertain behavior due to the lack of member section ductility, since stringent seismic detailing is not required in non-seismic zones. The response modification factor R (as employed in seismic analysis) is made up of two components: ductility, which is limited in wind engineering as mentioned above, and overstrength in the structure. Therefore, overloading beyond the peak strength or PMI limit is probably not feasible. However, overload beyond the FY strength limit, which is not currently allowed in building codes, might be investigated in an effort to develop a more economical design that not only satisfies life safety requirements, but also owner's expectations.

Another distinction in the design philosophy for earthquakes is that the ACI 318 (2002) building code section 21.4.2 attempts to deter story failure mechanisms by ensuring that the columns of a joint are stronger than the corresponding beams framing into the same joint by at least 20 percent. This requirement only applies to special RC moment frames. The intent is to limit column yielding on each story and to promote a more desirable beam sidesway mechanism where damage is distributed throughout the structure, as compared to being located primarily on one floor. Fig. 4 shows that in a column sidesway mechanism all columns of a particular story may simultaneously yield at a small inter-story drift level. This would imply that the primary source of overstrength for the structural systems is from material overstrength such as strain hardening. In contrast, strong column – weak beam systems possess significant structural overstrength and deformation capability prior to the development of a complete mechanism due to the spread of damage throughout the building. The lack of structural overstrength in non-seismically designed structures may lead to catastrophic behavior during overload conditions, since redistribution of forces is not possible.

Participation of Nonstructural Elements

As discussed previously, buildings designed in high-risk seismic zones are expected to undergo significant inelastic deformations during the design basis event. In the case for earthquake resistance, nonstructural participation can be detrimental to the overall building performance. Since nonstructural systems are typically force sensitive and have limited ductility, separation between the structural and nonstructural systems is required on the order of 2.0 to 2.5 percent of the story height to prevent nonstructural participation (ASCE-7, 2002).

In comparison for buildings in regions of low to moderate seismic risk, nonstructural cladding is typically isolated from the main structural framing by about 0.5 inches. As such, structural engineers generally limit interstory deformations during frequent wind event (typically between the 10-year and 50-year events) to 0.17 to 0.40 percent of the story height (depending on office practice) as compared to 2.0 to 2.5 percent in seismic zones. Therefore, during more intense wind loadings up to the design basis event (between the 10 and 700 year wind events), significant nonstructural participation may occur. This participation can significantly influence the overall structural stiffness and strength, but is typically not considered during the design process. So in effect, buildings designed for code-based wind loading may have additional overstrength and stiffness due to nonstructural participation. However, since nonstructural systems are force sensitive and have limited deformation capacity (ductility), the added strength and stiffness is significantly uncertain and unreliable in overload situations.

Load Effects

During earthquakes, lateral forces are induced in the lateral force resisting system by inertia of the building mass. In the general case of wind loading, wind pressures act on the building cladding and then are transferred to the lateral–force-resisting-system. In addition to these wind pressures, the building cladding may also be exposed to impacts from flying debris or missiles, both small and large. There are many documented cases of severe damage to the building cladding due to missile impact (Beason et al., 1984 and Beason and Lingnell, 2000). The resulting loads on the structural system can be significantly different after cladding failure. In one scenario, failure of a windward glass panel can cause a significant build up of internal pressure. This may lead to significant roof uplift pressures on the roof and induce roof failure if not properly tied down vertically. In another scenario, multiple glass panel failures may significantly decrease the wind loading on the lateral force resisting system by allowing the wind to pass somewhat unimpeded through the building. In this case, the structural system will remain intact, and however, significant damage to the building contents will result.

In an effort to protect building contents during wind loading, impact resistant glass has been introduced into the building architecture market. Impact resistant glasses typically considered in practice are laminated architectural glass (multiple glass panels with a Polyvinyl Butyral [PVB] interlayer) and filmed glass. These special glass panels are intended to remain intact and in contact with the structural system when exposed to overload pressures and missile impacts. By doing so, wind pressures can be eliminated from entering the building. Therefore, a potential issue in wind design using impact resistant glass is the fact that the lateral force resisting system may now be exposed to higher levels of wind loading, at least compared with those using standard architectural glass that are more vulnerable to failure during missile impact.

Another difference in PBSD vs. PBWD is with regards to the nature of the applied loading. Earthquakes tend to vibrate structural systems in a cyclic dynamic fashion. However, the response of low-to-mid-rise rigid structural systems during wind loading can be quasi-static in a monotonic fashion and can be dynamic in taller flexible systems.

It should be emphasized that the formation of well-detailed plastic hinges during earthquake loading may not be as important as hinges developed during static loading because of the inertia and hysteretic damping effects when responding dynamically. For statically applied wind loading, system response is more dependent on building system strength and stiffness, and not dependent on local member section ductility.

Finally, when the structure is loaded beyond the FY, the stiffness reduces and the period increases, which typically has beneficial effects on the structure during seismic loading. However, for most structures, when the period increases, the dynamic portion of the wind load increases. For tall structures, the dynamic portion of the wind can constitute a significant portion of the required loading demand. Therefore, there can be a tendency for the wind induced forces to increase due to stiffness reduction of the structure.

CONCLUSIONS

The paper provides an overview of the current design methodology for wind load based on ASCE-7 (2002) and IBC (2003). An attempt was made to identify the issues in developing a performance based design methodology for wind load. It was demonstrated that significant differences in performance based design methodologies for wind and earthquake loading will exist due to the nature of the loadings and the different design philosophies. Future research is required on establishing criteria for appropriate performance objectives and performance levels during serviceability and strength loading conditions.

ACKNOWLEDGMENTS

The author would like to acknowledge the contributions of ACI 375 committee members Viral Patel, Andres Lepage and Duane Castaneda for their thorough review of the paper and helpful suggestions to improve the quality of the manuscript. The author would also like to acknowledge Javier Horvilleur (deceased) of Walter P. Moore and Associates for involving me in ACI 375 committee activities and having detailed discussions on the topic of performance based design for wind loads.

REFERENCES

ACI 318-02 (2002). "Building Code Requirements for Structural Concrete and Commentary," American Concrete Institute, Farmington Hills, Michigan.

ASCE (2000). "Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA 356), prepared by American Society of Civil Engineers for the Federal Emergency Management Agency, Washington, D.C.

ASCE 7-02 (2002). "Minimum Design Loads for Buildings and Other Structures," American Society of Civil Engineers, Reston, Virginia.