



DESIGN DRIFT REQUIREMENTS FOR LONG-PERIOD STRUCTURES

Gary R. Searer¹ and Sigmund A. Freeman²

SUMMARY

The code provisions for calculating the design seismic drift of buildings have been substantially revised over the past 40 years. While these changes in the code are fairly well documented, the reasons behind these changes and the consequences of the changes are not as well known. This paper presents a brief history of design drift requirements, technical background for the requirements, and the reasoning behind the changes, starting with the 1961 Uniform Building Code (UBC) through present day.

Emphasis is given to the discussion of minimum base shears for calculation of drift for long-period structures. Specifically, in Section 1630.10.1 of the 1997 UBC, it is not immediately apparent why Equation 30-6 may be disregarded in the calculation of drift while Equation 30-7 may not, since both equations tend to give very similar minimum base shears for typical buildings. In prior versions of the UBC, the minimum design base shear was determined by only one equation that could be disregarded during determination of drift. This paper discusses the reasoning behind Equation 30-7 in the current UBC and discusses the current controversy and differences of opinion regarding this equation. Also discussed are equivalent requirements in the National Earthquake Hazards Reduction Program (NEHRP) and Minimum Design Loads for Buildings and Other Structures (ASCE 7-02), which require a similar minimum base shear for determining drift of long period structures.

Near-fault and non-near-fault earthquake records are analyzed to show the applicability of the use of these minimum base shears for determination of drift and suggested modifications to current building codes are presented.

INTRODUCTION TO DRIFT AND DEFLECTION

Lateral deflection is the predicted movement of a structure under lateral loads; and story drift is defined as the difference in lateral deflection between two adjacent stories. During an earthquake, large lateral forces can be imposed on structures; both the 1997 UBC (the basis of the 2001 California Building Code) and ASCE 7-02 (which is based on NEHRP) require that the designer assess the effects of this deformation on both structural and nonstructural elements. Lateral deflection and drift have three primary effects on a structure; the movement can affect the structural elements (such as beams and columns); the movements can affect non-structural elements (such as the windows and cladding); and the movements can affect

¹ Senior Associate, Wiss, Janney, Elstner Associates, Inc. Emeryville, CA, USA, gsearer@wje.com

² Principal, Wiss, Janney, Elstner Associates, Inc. Emeryville, CA, USA, sfreeman@wje.com

adjacent structures. Without proper consideration during the design process, large deflections and drifts can have adverse effects on structural elements, nonstructural elements, and adjacent structures.

Effect of Drift on the Structure

In terms of seismic design, lateral deflection and drift can affect both the structural elements that are part of the lateral force resisting system and structural elements that are not part of the lateral force resisting system. In terms of the lateral force resisting system, when the lateral forces are placed on the structure, the structure responds and moves due to those forces. Consequently, there is a relationship between the lateral force resisting system and its movement under lateral loads; this relationship can be analyzed by hand or by computer. Using the results of this analysis, estimates of other design criteria, such as rotations of joints in eccentric braced frames and rotations of joints in special moment resisting frames can be obtained. Similarly, the lateral analysis can also be used and should be used to estimate the effect of lateral movements on structural elements that are not part of the lateral force resisting system, such as beams and columns that are not explicitly considered as being part of the lateral force resisting system.

Design provisions for moment frame and eccentric braced frame structures have requirements to ensure the ability of the structure to sustain inelastic rotations resulting from deformation and drift. Without proper consideration of the expected movement of the structure, the lateral force resisting system might experience premature failure and a corresponding loss of strength. In addition, if the lateral deflections of any structure become too large, $P-\Delta$ effects can cause instability of the structure and potentially result in collapse.

Structural elements and connections not part of the lateral force resisting system need to be detailed to withstand the expected maximum deflections and drifts. Though these elements are generally ignored during the design lateral analysis, they must effectively “go along for the ride” during an earthquake, meaning that they experience deflections and rotations similar to those of the lateral force resisting system. Consequently, both the 1997 UBC and ASCE 7-02 require that the structural elements not part of the lateral force resisting system be designed to maintain support of design dead and live loads under the expected deformations, including any $P\Delta$ effects. One of the best examples of failure to ensure adequate deformation compatibility was the collapse of the parking garage at Cal State Northridge during the 1994 earthquake. The structure had ductile precast moment frame columns but lacked adequate deformation compatibility in the structural elements and connections that were not part of the lateral force resisting system, resulting in collapse of the interior gravity support system (EERI, 1994).

Effect of Drift on Nonstructural Elements

Since lateral deflection and drift affect the entire building or structure, design of nonstructural elements is also governed by these parameters. The nonstructural elements should be designed to allow the expected movement of the structural system.

If the nonstructural elements are not adequately isolated from the movements of the lateral force resisting system, adverse effects are likely. For example, in a large earthquake, the cladding may become damaged or fall off the structure, posing a life-safety hazard to passers-by. Even in smaller earthquakes, if the cladding does not permit lateral movement of the structure, the cladding may experience premature damage, resulting in water intrusion and/or economic loss. Similarly, if windows do not permit movement of the structure, the windows may break, posing a potentially significant falling hazard.

The effects of deflections and drift on stair assemblies are sometimes neglected. Without proper detailing that permits adequate interstory movement to occur, stair assemblies have the potential to act as a diagonal brace between floors; the stair assemblies resist the movement of the structural frame until damage to the stair assemblies or their connections occurs. If the vertical support for the stair assembly breaks or is

damaged, the stairs can collapse during the earthquake or even after the earthquake as the occupants attempt to exit.

Finally, if the nonstructural elements are not adequately isolated from the structural elements, the nonstructural elements may interfere with the structural elements and cause adverse effects to the structural elements themselves, creating short columns, torsion, or stiffness irregularities.

Effect of Drift on Adjacent Structures

Under lateral loads from a large earthquake, the expected movements of a structure can be significant. Consequently, both the 1997 UBC and ASCE 7-02 require that adjacent structures be isolated from each other by a prescribed distance so that contact between adjacent structures is minimized.

If adjacent buildings or structurally separate portions of the same structure do not have adequate separation, they may “pound” against each other during an earthquake. Pounding can have significant adverse effects, especially when the floors are not co-planar. Pounding of structures with non-co-planar floors can result in the floors of one building impacting the columns of another building at mid-height. This impact induces large shears and bending moments into the impacted columns, potentially causing the columns to fail and the structure to collapse.

When adjacent structures have coplanar floors, pounding may be advantageous in some respects. If floors are coplanar, the two adjacent structures will have a more difficult time resonating with the earthquake. Since pounding is a highly non-linear response, pounding will tend to damp out vibrations and reduce the responses of the two structures. However, the pounding is likely to increase floor accelerations (a consideration for the design of nonstructural elements) and is likely to result in significant localized damage between the structures.

A BRIEF HISTORY OF SEISMIC DRIFT

The following discussion documents how consideration of deflections and drifts was addressed by older codes as well as by current codes.

Older Codes

In 1961, the first deflection requirement was added to the Uniform Building Code (UBC), which required that buildings either be designed to act as an integral unit or be designed with sufficient separation to avoid contact under deflections caused by wind or seismic loads. Since the only seismic loads were based on an allowable design force, the standard of care at that time was to ensure that buildings would not touch under these loads. Engineers were also required to “consider” lateral deflections or drift of a story relative to its adjacent stories in accordance with “accepted engineering practice.”

In 1967, the first deformation compatibility requirements for exterior elements were added to the UBC. Connections of exterior cladding were required to allow for a relative movement of not less than two times the story drift caused by wind or seismic forces.

Deformation compatibility requirements were further extended in the 1973 UBC, which required that framing elements not part of the lateral force resisting system be adequate for vertical load-carrying capacity under four times the code-required lateral design forces.

The 1976 UBC further increased drift and deflection requirements by imposing a drift limit of 0.005 times the story height and requiring that the calculated drift be multiplied by a $1.0/K$ term, where K was analogous to the reciprocal of the more modern R or R_w factors. In addition, connections of cladding

elements as well as framing elements not part of the lateral force resisting system were required to be designed to accommodate movements of $3.0/K$ times the story drift caused by seismic forces. The option of calculating a given building's period using the Rayleigh Method (what was eventually defined as Method B in the 1988 UBC) was also added.

In 1988, the UBC underwent a dramatic change, switching from K 's to R_w 's, and modifying drift requirements. For structures under 65 feet in height, story drift was limited to $0.04/R_w$ or 0.005 times the story height. For structures over 65 feet in height, story drift was limited to $0.03/R_w$ or 0.004 times the story height. Exterior elements and framing elements not part of the lateral force resisting system were required to accommodate movements equal to $3(R_w/8)$ times the calculated deflections, and for the first time since 1961, the minimum building separations were modified by increasing the required separation to $3(R_w/8)$ times the calculated displacement due to seismic loads. Further complicating the requirements were the additions of a dynamic lateral force procedure, a minimum static force that limited C/R_w to 0.075 (effectively a minimum base shear coefficient of 3% for Zone 4 structures), limits on the static base shear resulting from the use of Method B period determination, and limits on the base shear resulting from the dynamic lateral force procedure compared to the static lateral force procedure. It is important to note that from the 1988 UBC through the 1994 edition, the minimum base shear and the limits on base shears resulting from the use of Method B were not required to be included when calculating seismic drifts and deflections.

Current Approach

By the 1997 UBC, R-factors had replaced the R_w -factors, near-fault effects had been added -- increasing design base shears by up to 100%, and the requirements for drifts and deflections had again changed dramatically. The replacement of R_w -factors with R-factors increased design forces by approximately 40% to conform to strength-based load combinations. The nominal displacements resulting from these strength-level forces were then defined as Δ_s . The maximum inelastic displacements, Δ_M , were then calculated by multiplying Δ_s by 0.7R. This had the effect of further increasing the deformations required for building separations, exterior elements, and structural elements not part of the lateral force resisting system. However, since allowable deformations were also increased by a proportional amount, it was thought that the effect of this change would be relatively minimal.

The increase in expected maximum inelastic displacements was adopted as a compromise between adherents of Newmark's so-called Equal Displacement Rule -- which stated that the maximum inelastic displacement of a structure can be approximated by the elastic displacement of the same structure under the unreduced earthquake -- and portions of the SEAOC membership who believed that the Equal Displacement Rule more often than not over-predicts displacements, or believed that requiring full consideration of the Equal Displacement Rule might be too onerous for designers, and/or believed that insufficient study had been done to justify the use of the Equal Displacement Rule for multi-degree-of-freedom systems. Thus, the 0.7 factor was selected as a compromise or "average" position between the two viewpoints (SEAOC, 1999).

In ASCE 7-02, the rules governing computation of maximum story drift appear somewhat more arbitrary, in that the designer calculates reduced seismic design forces but then scales up the calculated displacements by a C_d factor. However, instead of a constant factor similar to the 1994 UBC or a constant ratio of C_d to R similar to the 1997 UBC, the ASCE factor depends on the lateral force resisting system; and the ratio of C_d to R varies from 1.0 to 0.5, depending on the system. Although these values were originally proposed in ATC-3 in 1978, no explanation of how these factors were developed or chosen is given.

Story Drift Limitations

The 1997 UBC requires that story drift be limited to 0.025 for short period structures and 0.020 for long period structures; these or similar limits have been in place since the 1976 UBC. ASCE 7-02 requires that story drift be limited based on the type and use of the structure. The intent of both codes was to limit the interstory drift to a reasonable value, beyond which it was thought that the structure might experience loss of vertical stability. The UBC also allows these limits to be exceeded, provided that the greater drift can be tolerated by both structural elements and nonstructural elements that could affect life-safety. For the UBC, it is not clear why there are two limits on drift, one for short-period structures and one for long-period structures. In fact, since shorter period structures can have difficulty escaping from the constant acceleration region of the response spectrum, it could be argued that the prescriptive drift limits should be reversed, with longer period structures allowed larger drift limits.

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

Controversy Surrounding Equation 30-7

There has been significant debate and controversy surrounding UBC Equation 30-7. In the published version of the 1997 UBC, *two* minimum base shear equations are present (Equations 30-6 and 30-7), only one of which (Equation 30-6) is exempted from drift calculations. According to a three-page position paper published by SEAOC (Bachman *et al.*, 2001), SEAOC had also originally agreed to exempt Equation 30-7 from the drift requirements. However, due to an error, the published version of the UBC failed to exempt Equation 30-7. Out of a growing concern for near-field pulse effects on long-period structures, SEAOC then reversed its position and decided to support the use of Equation 30-7 in the UBC. Around this time, ICBO, the publisher of the UBC, realized that Equation 30-7 had not been exempted from the drift equations and issued an erratum that exempted Equation 30-7 from the drift limits. SEAOC then issued the three-page position statement that endorsed the use of Equation 30-7 for both near-fault and non-near-fault structures. However, the justification for this position statement was a comparison of a few extremely large, near-field earthquake response spectra with the non-near-field, Soil Type C design spectrum. The position statement concluded that since the large, near-field earthquake response spectra exceeded the non-near-field, Soil Type C design response spectrum, the use of Equation 30-7 for drift calculations was needed to control drift. However, the comparison and the conclusions were arguably flawed, since the near-field records should have been compared with a near-field response spectrum with Soil Type D, which would have increased the design spectrum size in the longer periods by a factor of more than two. Figure 1 shows the response spectra from the five earthquake records that were used in the SEAOC position paper and compares these spectra with several code design spectra.

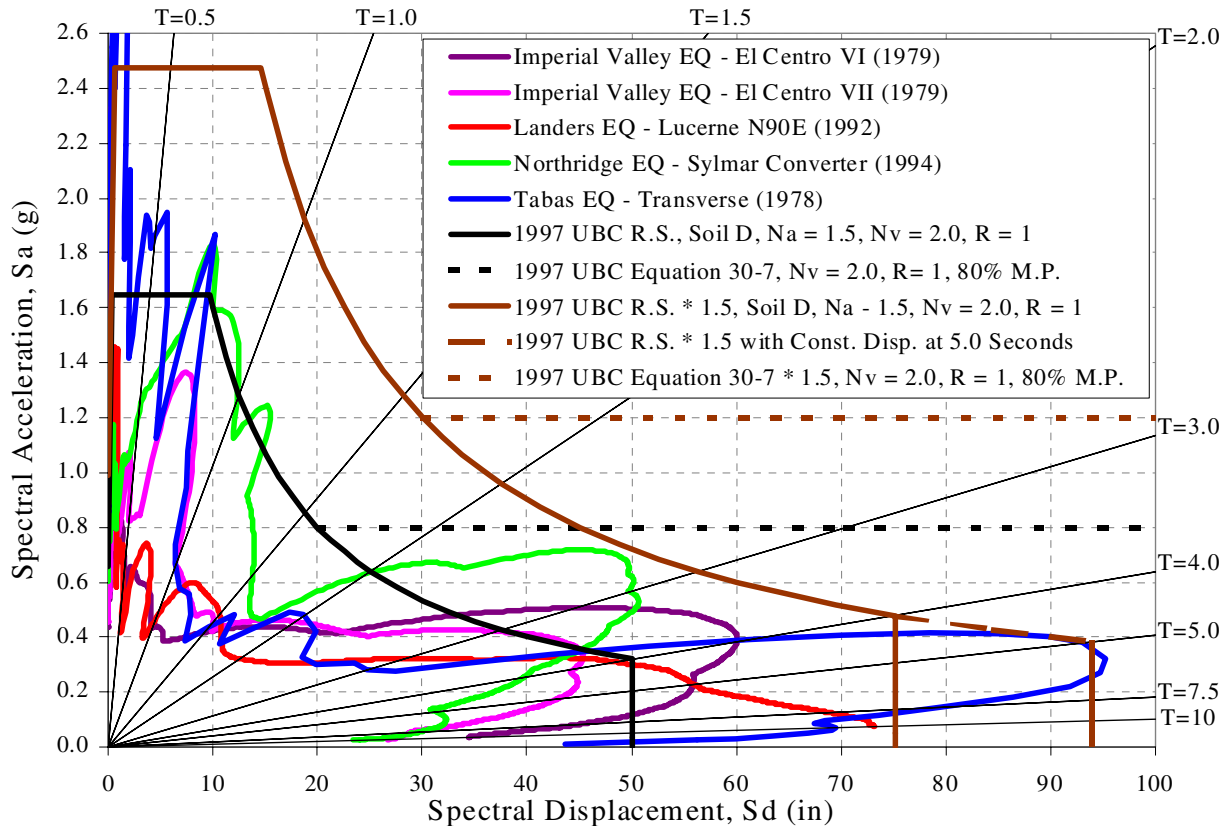


Figure 1. Comparison of five near-field earthquakes with the UBC near-field response design spectrum.

Compared to the 1997 UBC response spectra with Soil Type D, $N_v = 2.0$, and an R of 1.0, the actual earthquake response spectra exceed the design response spectrum in all three portions of the design response spectrum (i.e. constant acceleration, constant velocity, and constant displacement), which provides no specific impetus for a minimum base shear equation to address the constant displacement region of the spectrum alone. Furthermore, when the earthquake response spectra are compared against the equivalent of the Maximum Considered Design Earthquake (1.5 times the 1997 UBC design response spectrum), the earthquakes are essentially contained within the design envelope. Assuming that the response spectrum from the 1978 Tabas earthquake is actually accurate in the long period regions, it could be argued that the constant displacement cut-off should start at 5.0 seconds instead of 4.0 seconds. Either way, it is clear from the figure below that the minimum base shear from Equation 30-7 (assuming Soil Type D, an R of 1.0, and 80% mass participation) greatly over-shoots any demands from these near-field earthquakes. If adjusted for the MCE event (by multiplying the demand by 1.5), the disparities between Equation 30-7 and real earthquake demands becomes even larger. Thus, consideration of minimum base shears in computing deflections does not appear justified at this point.

APPLICATION OF THE CODE IN REAL LIFE

The drift provisions in the 1997 UBC and ASCE 7-02 are fairly complicated. Implementation of these provisions has significant benefits and yet poses substantial difficulties to designers.

Benefits

Assuming that the drift provisions in the UBC and ASCE 7-02 can indeed be followed, the benefits of successfully implementing the provisions can be substantial.

If properly designed, structural framing elements and connections not part of the lateral force resisting system should remain largely intact and be able to provide vertical support for the structure even at large deflections. This eliminates non-ductile failure of vertical load resisting system, such as that seen by some parking garages during the 1994 Northridge Earthquake.

Non-structural exterior elements will remain largely undamaged by earthquake ground motion. This will help prevent falling hazard and will limit water intrusion after an earthquake.

Finally, if the drift provisions in current codes are followed buildings should generally not pound together, which should reduce the threat to the vertical stability of columns from impact from adjacent structures' non-co-planar floor diaphragms and should eliminate localized pounding damage at co-planar diaphragm locations.

Difficulties

Recent studies (Freeman and Searer, 2000) have found that the drift provisions in the 1997 UBC are extremely complicated, are fairly difficult to use, and may be overconservative. Furthermore, it has been shown that improper application of the provisions can result in significant additional overconservatism in the design of structural and nonstructural elements.

It can be very difficult to ensure that exterior elements conform to the drift requirements in current codes (SEAOC, 1999). For a 13-foot story height, design interstory drifts can be as large as approximately 4-inches in any direction. If slotted holes are to be provided to allow this drift, the slots would need to be in excess of 8-inches long. By the time construction tolerances are accounted for, the required slots would need to be 9- or 10-inches long. Claddings with articulations (corners) have proven extremely difficult to isolate from lateral movements, since slotted holes generally only work for planar cladding elements. Even if a cladding manufacturer can design a cladding that conforms to the drift requirements, improper construction can quickly negate that effort (Searer and Freeman, 2004).

Windows can be substantially more difficult to assess. Current technology is often not adequate to accommodate racking on the order of 4-inches in each direction (for a 13-foot story). Even if a window manufacturer can provide calculations that show that a cladding will accommodate drifts on the order of 2.5 percent, use of shims and blocks during actual installation of the windows can often preclude movement from occurring.

In order to conform to the drift requirements, sealant joints (between cladding elements or between various portions of structures) can become extremely large, which can have adverse effects on the performance of waterproofing for the structures.

Finally, it is not entirely clear how the most common type of all structures in the United States -- the wood-frame structure -- should be designed with respect to drift requirements. In general, the standard of practice for wood-frame designers has been to ignore the code drift requirements for wood-framed residential structures, which have substantial nonlinear behavior at even low loads, and which are generally designed without the use of computer models.

RECOMMENDATIONS

Based on the above discussion, it appears that a more restrictive drift limit for longer-period structures as is currently the case in the 1997 UBC is unwarranted, since low-rise and mid-rise structures have historically performed far worse in the U.S. than modern high-rises. In a study of sixteen different structures (Freeman and Searer, 2000), the structure that had the greatest drift demands and would arguably be the most likely to collapse during a large earthquake was the 3-story steel moment frame and not the high-rises that were analyzed. In fact, conventional structural engineering wisdom dictates that the best place to be in a major earthquake (other than an open field or possibly in a base-isolated structure) is in a high-rise. This confidence in tall, long-period structures is due to the ability of the longer-period structures to accommodate relatively large displacement demands with relatively small displacements on each floor. For example, if a 50-story structure experiences 1.5-inches of displacement per floor (only 1% story drift assuming 13-foot high stories), that displacement equates to a 50-inch spectral displacement demand. If the same structure can withstand say twice that amount, i.e. 2% story drift, that equates to a spectral displacement capacity of 100-inches, more than sufficient to ride out even the largest recorded earthquakes, possibly without even experiencing inelastic behavior. Finally, as stated earlier, since shorter period structures can have difficulty escaping from the constant acceleration region of the response spectrum, it appears that reversal of the 1997 UBC prescriptive drift limits may be warranted, with shorter-period structures allowed smaller drift limits and longer-period structures allowed larger drift limits. By increasing the stiffness of low-rise structures, the strength of these structures is generally increased, and ductility demands on these structures as they struggle to escape the constant acceleration region of the response spectrum are generally decreased.

Some building codes limit interstory drift to much smaller values. For example, in Peru, drift is limited to 0.7% for reinforced concrete structures, 1.0% for steel structures, 0.5% for masonry structures, and 1.0% for wood structures, based on the full, unreduced earthquake demand (National Construction Code, Peru, 1997). Consequently, drift requirements are generally accomplished by the addition of reinforced concrete or confined masonry shear walls. There are definite advantages to using a strong and stiff lateral system, such as reinforced concrete shear walls, since structural damage can be limited to cracking and spalling of the shear walls, which protects the vertical load resisting elements from significant damage. Damage to nonstructural elements such as partitions, walls, cladding, and windows will be similarly limited. Floor accelerations would be increased, which could increase damage to certain other nonstructural elements and equipment unless they were also designed and constructed for larger forces. If U.S. public policy makers or code development agencies one day decide that reduction of damage and economic loss due to earthquakes is a significant priority, reducing the allowable story drift for short period structures and requiring a corresponding increase in design force requirements for nonstructural elements might be a fairly simple way to accomplish this. However, it is important that artificial limits and minimum base shears for the calculation of displacement be eliminated, so that the best approximations of displacement and drift can be obtained.

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