A review of conceptual transparency in US and Colombian seismic design building codes

Revisión de la transparencia conceptual en los reglamentos de diseño sísmico de edificios en EU y Colombia

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ABSTRACT

The goal of re-examinination of seismic design codes is aimed at making their provisions as transparent as possible for users so that the design would be clear and enriching for structural engineers. This paper presents a transparency evaluation of the codes currently being used in the USA and Colombia for seismic design of buildings. It is demonstrated that the procedures used in most codes do not offer a clear view of buildings' seismic response assessment. The Colombian code should become as conceptually transparent as possible when defining strength modification factors and assessing maximum lateral displacement. In addition, at least two limit states (service and life safety) should be clearly defined, along with allowable story drift thereby better reflect expected structural performance. Otherwise, using current procedures could lead not only to interpretation errors but also inadequate estimation of seismic strength and deformation demands.

Keywords: seismic design, strength reduction, over-strength, ductility, displacement amplification, drift limit.

RESUMEN

La reexaminación de los reglamentos de diseño sísmico pretende que los requisitos sean tan transparentes como sea posible para los usuarios, de tal manera que el proceso de diseño sea claro y enriquecedor para los ingenieros estructurales. En este artículo se presenta una evaluación de la transparencia de los reglamentos utilizados actualmente en Estados Unidos y Colombia para diseño sísmico de edificios. Se demuestra que los procedimientos utilizados en la mayoría de los reglamentos de edificios no proporciona una visión clara para evaluar la respuesta sísmica. El reglamento colombiano debe llegar a ser tan conceptualmente transparente como sea posible, en cuanto a la definición de los factores de modificación de resistencia y a la evaluación de los máximos desplazamientos laterales. Adicionalmente, se deben definir claramente mínimo dos estados límite (servicio y seguridad a la vida), junto con derivas de piso permisible que reflejen mejor el comportamiento estructural esperado. De lo contrario, el uso de los procedimientos vigentes podría originar no solo errores de interpretación, sino estimación inadecuada de las demandas de resistencia y deformación.

Palabras clave: diseño sísmico, reducción de resistencia, sobrerresistencia, ductilidad, amplificación de desplazamiento, límite de deriva.

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Introduction

In the current seismic design codes of many countries, base shear is calculated by elastic strength demand divided by a strength reduction factor. This factor reflects the influence of the structure's elastic-plastic deformation and energy-dissipating capacity (i.e. reduced forces due to nonlinear hysteretic behaviour). A displacement amplification factor is used to compute the expected maximum inelastic displacement from the elastic displacement induced by the design seismic forces (Uang, 1989).

The assessment of the minimum lateral strength capacity resulting in suitable control of inelastic deformation during strong earthquake ground motions requires a good estimation of the strength reduction factors. These reduction factors have been the topic of several investigations over the last 40 years. However, many of these investigations' findings so far have not been incorporated into building codes. Several researches have expressed their concern about the lack of rationality regarding the reduction factors currently specified in building codes (Rojahn, 1988). The improvement of reduction factors has been identified as a way of improving the reliability of current earthquake-resistant design provisions (Miranda and Bertero, 1994).

This paper was thus aimed at showing US and Colombian earthquake-resistant codes as transparent as possible for the users, so that their design will be clearer for structural engineers. This paper thus provides an overview of the development and the

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most relevant changes in US and Colombian earthquake-resistant design codes, compares and discusses the seismic-design approaches specified by these codes and the challenges involved in improving code compliance, particularly the Colombian code. The study includes a discussion of the most important parameters for seismic design, such as strength modification and displacement amplification factors. The main components needed to calculate these factors, such as the structural over-strength factor and structural ductility ratio, are also discussed. A comparison between codes concerning drift limit and reduction factors is also discussed.

Strength modification factors

Design lateral strengths prescribed in earthquake-resistant design provisions are typically lower, in some cases much lower, than the lateral strength required for maintaining a structure in an elastic range in the event of severe earthquake ground motions. Strength modifications from the elastic strength demand are commonly accounted for using both reduction and amplification factors (Miranda, 1997).

Reduction factor due to nonlinear hysteretic behaviour

A typical idealisation of the structural response is shown in Figure 1a. The level of inelastic deformation experienced by the system experiencing a given ground motion is typically given by the displacement ductility ratio μ (Priestley, 2000). Idealising the actual structural response curve by the linear elastic-perfectly plastic curve in Figure 1a, then structural ductility ratio can be defined as the ratio of maximum relative displacement to its yield displacement (Miranda and Bertero, 1994).

$$\mu = \frac{\Delta_{max}}{\Delta_y} \qquad (1)$$

Figure 1a also shows the required elastic strength expressed in terms of maximum base shear developing in a structure if it was to remain in the elastic range V_e. Since a properly-designed structure can usually provide a certain amount of ductility, then such structure is able to dissipate hysteretic energy. Because of such energy dissipation, a structure can be designed economically and thus, elastic design force V_e can be reduced to yield strength level V_y, by factor R_μ (Figure 1a) (Moroni *et al.*, 1996); the corresponding maximum deformation demand is Δ_{max}:

$$V_y = \frac{V_e}{R_\mu} \qquad (2)$$

Since calculating V_y and Δ_{max} involves nonlinear analysis, these quantities are not usually explicitly quantified. Strength reduction factor R_μ (i.e., the reduction in strength demand due to nonlinear hysteretic behaviour) is one of the first and most-studied reduction factor components. Factor R_μ is defined as the ratio of elastic strength demand to inelastic strength demand (Miranda, 1997):

$$R_\mu = \frac{V_e}{V_y} = \frac{F_y(\mu = 1)}{F_y(\mu = \mu_i)} \qquad (3)$$

where $F_y(\mu=1)$ is the lateral yielding strength required to keep the system elastic and $F_y(\mu=\mu_i)$ is the lateral yielding strength required to keep displacement ductility ratio demand μ, less than or equal to a predetermined maximum tolerable displacement (target) ductility ratio μ_i, when subjected to the same ground motion (Miranda and Bertero, 1994). A 5% equivalent viscous damping ratio is usually considered when computing reduction factor R_μ (Uang, 1989).

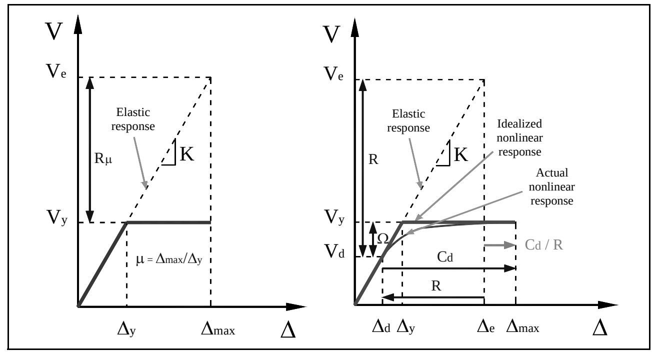

Figure 1. Structural response: (a) idealised, (b) overall

Studies reviewed by Miranda and Bertero (1994) agreed that for a given ground motion (i.e. ground acceleration time history), reduction factor R_μ is primarily influenced by the level of inelastic deformation (i.e. displacement ductility ratio μ), the natural period of the structure *T,* and soil conditions at the site.

It is worth noting that strength reduction factor R_μ prescribed by US codes (NEHRP-03, IBC-09, ASCE 7-10) and by some Latin-American codes, such as the Colombian code (NSR-10) disregard the period of vibration, which is incorrect, and thus their use is not recommended (Miranda, 1997).

Amplification factor due to over-strength

As well as R_μ, another strength modification can be considered in the design to take over-strength into account. Over-strength did not enter into the previous discussion because structural response was considered to be an idealised system.

There are several sources of structural over-strength. Most are related to the sequential yielding of critical regions, internal force redistribution (redundancy), actual materials strength higher than those specified in design, strain hardening of reinforcing steel, capacity reduction factors ϕ, member selection (member oversize), minimum requirements by codes regarding proportioning and detailing, multiple loading combinations, deflection constraints on system performance, level of force redistribution taking place in the structure, the effect of non-structural elements and strain rate effect (Uang, 1989; FEMA-451).

Figure 1b shows the typical overall structural response. The actual structural response, the idealised linear elastic-perfectly plastic response and elastic response are included in the figure. The structure remains essentially elastic until the first full plastic hinge forms. This level is commonly called the "first significant yield", i.e. the level beyond which global structural response starts to deviate significantly from elastic response.

The first significant yield is the level of force that causes complete plastification of at least the most critical region of the structure (e.g. first plastic hinge formation). The formation of this "first significant yield" occurs at a load level referred to as system design strength, V_d. The reserve strength between actual structural yield level and code-prescribed first significant yield V_d, is usually defined in terms of the over-strength factor Ω. As shown in Figure 1b, over-strength factor Ω can be defined as the ratio between V_y and V_d, the latter being the required strength prescribed by codes using a strength design approach (Moroni *et al.*, 1996).

$$\Omega = \frac{V_y}{V_d} \quad (4)$$

For design purposes, NEHRP-03 reduces V_y level to V_d level, the latter corresponding to first plastic hinge formation. The advantage of specifying V_d as the design level is that designers need only perform an elastic structural analysis. The first problem associated with this type of "elastic" design procedure is that designers do not know the true strength of the structure. If the reserve strength of a structure (the so-called over-strength) beyond design level V_d is significantly less than that implicitly assumed in the seismic provisions, then structure performance is not likely to be satisfactory during severe earthquakes. The second problem is that the maximum inelastic displacements cannot be calculated from elastic analysis results (Uang, 1989).

Deflection amplification factor

The equal displacement approach

Parameter μ has been widely accepted as a useful performance indicator because of its apparent relationship to strength reduction factor R_μ. The equal displacement concept is the basis for dividing "elastic" force demands by a strength reduction factor. It is one of the most important concepts in earthquake-resistant design. It implies that "the displacement of an inelastic system, having stiffness K and strength V_y, subjected to a particular ground motion, is approximately equal to the displacement of the same system responding elastically" (system displacement is independent of system yield strength) (FEMA-451). As shown in Figure 1a, the equal displacement approach of seismic response implies that (Priestley, 2000):

$$\mu = R_\mu \quad (5)$$

Building codes consider elastic structural analysis based on applied forces becoming reduced to account for the presumed ductility supplied by the structure (based on the level of detailing provided). Using reduced forces from elastic analysis will result in a significant underestimate of displacement demands. Therefore, displacements arising from reduced-force elastic analysis must be multiplied by the ductility ratio to produce true "inelastic" displacements.

It has been shown that an equal displacement approach is non-conservative for short period structures, roughly corresponding to the first region of the spectrum. Equal energy approach should be applied in this region. This reduction is lower and depends on both period of vibration T and ductility capacity μ. The primary reason is that short period systems tend to display significant residual deformations. Thus, R_μ increases linearly in the first region of the spectrum from R_μ= 1 to a value close to ductility ratio μ. ASCE 7-10 effectively reduces the acceleration spectrum by a strength reduction factor in all period ranges (FEMA-451). However, the ASCE 7-10 code allows no reduction of peak ground acceleration in the very short period region (acceleration spectrum having a constant plateau extending from T = 0 s) so this partially compensates for error in equal displacement assumption at low period values.

In the spectrum's mid-region, R_μ is only slightly dependent on period of vibration T. However, it is of doubtful validity for medium period structures when the hysteretic nature of the inelastic system deviates significantly from elastic-perfectly plastic. For very long periods, the R_μ factor maintains a constant value equal to prescribed ductility μ, and thus the equal displacement approach can be applied ($R_\mu = \mu$)(FEMA-451). According to Newmark and Hall (1982), for structures having long, medium and short periods, $R_\mu = \mu$, $R_\mu = (2\mu - 1)^{0.5}$, and R_μ = 1, respectively. These expressions indicate that R_μ / μ is not greater than 1. Moreover, this ratio is significantly less than 1 for structures having medium and short periods.

Displacement amplification

Displacements from elastic analysis involving reduced forces are amplified by the displacement amplification factor C_d to estimate the structure's maximum expected displacements, including effects caused by inelastic deformation. Factor C_d is defined as the ratio between maximum expected nonlinear displacement during an earthquake Δ_{max}, and elastic displacement induced by reduced seismic forces Δ_d (Moroni *et al*., 1996)(Figure 1b).

$$C_d = \frac{\Delta_{\max}}{\Delta_d} \quad (6)$$

The displacement amplification factor C_d can also be derived from Figure 1b as follows:

$$C_d = \frac{\Delta_{\max}}{\Delta_d} = \frac{\Delta_{\max}}{\Delta_y}\frac{\Delta_y}{\Delta_d} \quad (7)$$

where Δ_{max}/Δ_y is μ, and Δ_y/Δ_d from Figure 1b is:

$$\frac{\Delta_y}{\Delta_d} = \frac{V_y}{V_d} = \Omega \quad (8)$$

Therefore, (7) can be expressed as:

$$C_d = \mu\,\Omega \quad (9)$$

From these derivations, it can be observed that the C_d factor is a function of structural over-strength factor, structural ductility ratio and damping ratio; damping effect is usually included in ductility reduction factor R_μ.

Evaluating building codes

The evolution and practice of US and Colombian seismic codes is briefly described and discussed in this section.

US codes

Figure 2a shows procedures for seismic design prescribed by US building codes, such as NEHRP-03, IBC-09 and ASCE 7-10.

Strength modification factors: In US process, design seismic forces are obtained by reducing a linear elastic response spectrum by response modification factor R and then member forces are determined through linear elastic analysis. Hence, factor R is defined as the ratio between base shear in the structure if it was to remain in the elastic range and the minimum base shear required to resist seismic action and to accommodate nonlinear displacements without any risk to its stability (Moroni *et al*., 1996). Figure 2a shows that total strength modification factor R can be considered the product of ductility reduction factor R_μ and structural over-strength factor Ω (Varela *et al*., 2004):

$$R = \frac{V_e}{V_d} = R_\mu \Omega \quad (10)$$

Equation 10 shows that it is misleading to call R the ductility reduction factor, because structural over-strength may play a role equally important than ductility in R factor (Uang, 1989). Similarly to C_d, R prescribed in seismic codes is primarily intended to account for energy dissipation capacity and over-strength; however, it also accounts for damping (if different from 5% of critical damping) and redundancy.

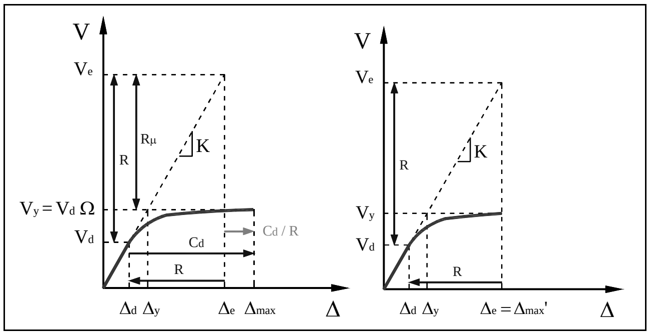

Figure 2. Procedures in building codes: (a) US, (b) Colombia.

Factors R and C_d usually depend on the period of the structure, structural system type and the structural ductility. However, R and C_d prescribed in US seismic codes are primarily based on the observation of the performance of different structural systems in previous strong earthquakes, consensus of engineering judgment, technical justification and tradition.

Strength reduction factors are one of the most controversial aspects of current building codes. Several researchers (Uang, 1989; Miranda and Bertero, 1994) have expressed their concern about the lack of rationality in current R factors and their improvement has been identified as a way to improve the reliability of present earthquake-resistant design provision. For instance, most investigations reviewed by Miranda and Bertero (1994) recommend the use of period-dependent strength reduction factors. Uang (1989) established basic formulas for evaluating R and C_d from global structure response characterised by the relationship between base shear ratio and storey drift. Variations in R_μ, with changes in period of vibration, are not incorporated in current seismic provisions for building structures in the US. The permissible levels of strength reduction are only based on the type of structural system.

Current seismic design provisions in the US do not require designers to quantify R and Ω factors. ASCE 7-10 provides the R and Ω_0 factors for a large number of structural systems. Tables 1 and 2 show the design coefficients for a few selected concrete and steel systems, respectively.

Table 1. Design factors in ASCE-7-10 for concrete structures

Structural system	R	Ω_0	$R_\mu=R/\Omega_0$	C_d
Special moment frame	8.0	3.0	2.7	5.5
Intermediate moment frame	5.0	3.0	1.7	4.5
Ordinary moment frame	3.0	3.0	1.0	2.5
Special reinforced shear wall	5.0	2.5	2.0	5.0
Ordinary reinforced shear wall	4.0	2.5	1.6	4.0
Detailed plain concrete wall	2.0	2.5	0.8	2.0
Ordinary plain concrete wall	1.5	2.5	0.6	1.5

It is very important to note that R_μ is ductility demand only if Ω_0 is achieved and "ductility demand" R_m is minimum because Ω_0 as listed in the tables is the "maximum expected over-strength. A ductility demand equal to one or less indicates that the "expected" response for these systems is essentially elastic.

Constant R and C_d factor values do not ensure the same level of safety against collapse for all structures. For buildings having minimal redundancy, structural over-strength relied upon by current seismic design provisions may be insufficient. There is a need for incorporating a method to quantify the structure's over-strength; such over-strength should not be less than that assumed in establishing R and C_d (Uang, 1989).

Table 2. Design factors in ASCE-7-10 for steel structures

Structural system	R	Ω_0	$R_\mu=R/\Omega_0$	C_d
Special moment frame	8.0	3.0	2.7	5.5
Intermediate moment frame	4.5	3.0	1.5	4.0
Ordinary moment frame	3.5	3.0	1.2	3.0
Eccentric braced frame	8.0	2.0	4.0	4.0
Eccentric braced frame (pinned)	7.0	2.0	3.5	4.0
Special concentrically braced frame	6.0	2.0	3.0	5.0
Ordinary concentric braced frame	3.3	2.0	1.6	3.3
Not detailed	3.0	3.0	1.0	3.0

Even though the equations presented by Miranda and Bertero (1994) seem reasonable and may be incorporated in future US seismic codes, today (2013) single values for R are still proposed in such seismic codes for designing different structural systems.

Displacement amplification factor: based on the equal displacement approach, inelastic displacement demand is the same as elastic displacement demand. Figure 2a shows clearly that, displacement Δ_d predicted by this analysis would be too low. "Computed design displacement" Δ_d should be multiplied by displacement modification factor C_d and thus to obtain an estimate of true maximum inelastic response to correct for the too-low displacement predicted by the reduced force elastic analysis. This factor is always less than R because R contains ingredients other than pure ductility.

Similarly to R and Ω factors, ASCE 7-10 provides C_d (see Tables 1 and 2). It is interesting to examine the ratio C_d/R in Figure 2a. It can be shown from (9) and (10) that:

$$\frac{C_d}{R} = \frac{\mu\,\Omega}{R_\mu\,\Omega} = \frac{\mu}{R_\mu} \tag{11}$$

Equation 11 indicates that the C_d/R ratio for a particular structural system is a function of structural ductility ratio only through R_μ and μ, and is independent of the structural over-strength factor (Uang, 1989). C_d/R ratios specified by US codes are constant and independent of the period of vibration, thus estimating inelastic displacements is not suitable for structures having a short period and resting on rock or on firm soil. For structures resting on soft soils, the estimate is adequate only for structures having a very long period ($R_\mu = \mu$).

ASCE 7-10 also provides allowable story drift to be compared to true maximum inelastic drift. As shown in Table 3, allowable drift depends on a building's importance.

Colombian code

Many areas of South America are noted for their high seismicity. Recognising the region's seismic activity, earthquake-resistant design of structure is thus a requirement in these countries. Therefore, each country has developed its own seismic code based on their experience and laws. The codes also follow aspects of UBC-97and IBC-09.

Table 3. Story drift limits in ASCE 7-10

Structural system	Risk category		
	I or 2	3	4
Structures, other than masonry wall structures, 4 stories or less above the base with partitions that have been designed to accommodate story drift	2.5 %	2.0 %	1.5 %
Masonry cantilever shear wall structures	1.0 %	1.0 %	1.0 %
Other masonry shear wall structures	0.7 %	0.7 %	0.7 %
All other structures	2.0 %	1.5 %	1.0 %

Most South-American codes' traditional design philosophy is to maintain life safety by avoiding collapse during severe earthquakes. Although different activity levels may be used, the design earthquake is typically an event having a 475-year return period, as used in UBC-97.

Strength modification factors: the overall seismic design procedure prescribed by the 1998 edition (NSR-98) and by the recently released NSR-10 is shown in Figure 2b.

Colombian codes have used the conventional force-based/displacement-check approach. The 1984 Colombian Seismic Code used a response modification factor R that varied for each structural system, material and seismic risk (Garcia, 1996). R is the reduction factor used by NSR-98 and NSR-10 codes to decrease the elastic seismic forces. The reduction factor in NSR has the same purpose as in the US codes (Eq.10), i.e. to account for the global ductility capacity of the lateral force resisting system R_{μ}, and the over-strength inherent in lateral force resisting system Ω. R is a function of the type of the system, period of vibration, irregularity and a building's expected design level or design category. R in NSR-10 is also a function of structural system redundancy. The Colombian seismic code uses reduction factors ϕ (always < 1) to account for any irregularity and redundancy in the structure. In the 1984 edition, R was a single and constant value used to constantly reduce elastic forces, regardless of a structure's period of vibration (Chavez, 2012). R prescribed by NSR-98 and NSR-10 codes does not vary with period of vibration when the code spectrum is used, and it does so only for the micro-zoning spectrum.

The recently released NSR-10 also explicitly specifies an over-strength factor Ω_0 related to the seismic-force-resisting system and is used for designing certain fragile elements which are incapable of dissipating energy in the non-lineal range, such as certain wall piers, anchors and collector elements, or where there are greater concerns about shear failure. For designing such elements, the design shear force need not exceed Ω_0 times the factored shear determined by analysing the structure for earthquake effects. Amplification factor Ω_0 ranges from 1.5 and 3.0, depending on the type of seismic system.

The approach involving using amplification factor to account for the seismic-force-resisting system's over-strength has been adapted from the ACI 318-11 Building Code, where design shear force is computed as Ω_0 times the shear induced under design displacements.

The effect of over-strength should be accounted for when evaluating a member's strength (as an amplification factor regarding strength). Because of the limitations of using advanced non-linear analysis techniques by practicing engineers, most building codes apply the effect of over-strength as a reduction factor to the loads instead of an amplification factor to the strength. However, the NSR-10 approach could be doubtful because it attempts to amplify earthquake forces by Ω_0, instead of amplifying member strength or reducing earthquake loads.

Displacement evaluation: in the 1984 Colombian Seismic Code, drift was obtained from elastic deflections amplified by deflection amplification factor C_d. This factor also depended on the structural system, material and seismic risk level. The allowable drift limit was a single constant value equal to 1.5%. If drift was within the allowable limits, the designer could design the different elements using the requirements for each seismic risk level. If the drift requirements were not met, a structure had to be stiffened and re-analysed (Garcia, 1996).

According to NSR-10, structures can sustain extensive damage without collapsing when subjected to the design earthquake; this implied that a collapse prevention limit state was adopted. The NSR-10 approach seems to assume the equal displacement approach (Figure1a) because "inelastic" displacement Δ_{max}' is equal to the displacement which would occur during elastic response Δ_e (Figure 2b). Allowable drift for masonry structures controlled by shear deformations is 0.5%; for other structural systems the drift limit is 1.0%. A 1.0% drift limit is required when gross section stiffness is used in analysis; if cracked sections are used in analysis, calculated drift must be reduced by 30% before comparison (Restrepo, 2008).

Allowable drifts prescribed in NSR-10 are very different from the values prescribed in forming codes (see Table 3). When comparing a 1.0% drift limit in NSR-10 with 2.0% or 2.5% for the collapse prevention limit stated in ASCE 7-10, the drift limits presented in NSR-10 are more related to serviceability than collapse prevention limit state, regardless of the text within the code referring to this scenario as a collapse prevention limit state. The 1984 seismic code specified a 1.5% drift limit; this value was later reduced to 1.0% in NSR-98. The reason for such reduction was to prevent non-structural damage and to encourage the use of shear walls due to the good performance achieved during an M_w=7.8 earthquake in Chile in 1985. Some studies were carried out for making such change, but they were mostly based on financial loss (Garcia, 1996) without paying attention to changes in reduction factors that such modification would have created. Trying to protect non-structural elements is a very important issue in a country like Colombia having limited economic means, as pointed in the above studies. The root of the problem lies in trying to prevent non-structural damage and conceive a ductile structure in the same scenario. The serviceability limit state seems to be coherent with the drift limits prescribed within the code and collapse prevention limit state seems to be coherent with reduction factor R suggested in the code.

Based on the roughly elastic behaviour found with time history analysis (THA) of reinforced concrete frames, Restrepo (2008) concluded that the way that seismic design was being used in NSR-98 (similar to NSR-10) seemed to be inappropriate as it led to very expensive RC frame structures and structures whose performance was quite beyond requirements. The main reason of such trend takes root in the single scenario that NSR-98 (and NSR-10) used to perform structural analysis. Member sections required to satisfy drift limits are immense, making very important the minimum steel requirements given by the code, as this imposes high ductility detailing to a structure which will perform almost elastically. Such elastic performance raises large doubts about the reduction factors used in design which are mainly based on the supposed inelastic movements that the structure will undergo when subjected to the design earthquake.

The Colombian code has adapted the US codes without making significant changes related to members design, structural system, analysis methods and/or hazard analysis; however, a major change was made in the analysis methodology since the Colombian code uses only one scenario for designing both structural and non-structural elements. This change to only a single design scenario comes with drift limits which are inconsistent with both serviceability and ultimate limit states. The drift limits prescribed in NSR-10 are very different from the values prescribed in the US forming codes. An inconsistency arises by using a high reduction factor R = 7.0 having a 1% restrictive drift limit for concrete frames. For instance, it does not seem logical to use the drift values prescribed in NSR-10 with some of the considerations for ductile structures within the code (Restrepo, 2008). This type of stagnation is not consistent with that found in other codes around the world.

Conclusions

This paper has summarised and discussed the approach adopted in seismic design provisions for buildings in the USA and Colombia. The following conclusions can be drawn from this study:

Strength modification and displacement amplification factors (empirical to date) have been mainly based on engineering judgment consensus and observed structural performance during previous earthquakes. The only way to rationalize these factors is to quantify over-strength and structural ductility ratios by analytical studies and experimental testing. The use of rational strength modification and displacement amplification factors based on ductility, period and soil conditions, together with estimates of the structure's over-strength and the relationship between global and local ductility demands are needed to establish a more rational and transparent seismic design approach than that currently being used in the Colombian code.

In addition, more rational criteria need to be stipulated in NSR-10 for computing lateral displacements. It is recommended that a performance-based design approach be included in the Colombian code to include at least two limit states based on specific return periods. Each limit state should include specific drift limits considering the type of non-structural elements attached to the structure and a particular structural system.

Assessment of Colombian codes (Restrepo, 2008) has shown discrepancies regarding performance requirements and safety levels. A serious effort ought to be made to improve such codes and their enforcement. For instance, the procedure prescribed in the next edition of the Colombian building code should allow determining design strengths and displacements in a more rational way, more in accordance with the present state of knowledge and contemporary tendencies in building codes.

Despite these criticisms, it should be noted than current force-based seismic design, when combined with capacity design principles and careful detailing, usually produces safe and satisfactory designs. However, the degree of protection provided against damage under given seismic intensity is non-uniform from structure to structure.

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