

### THE NEED OF PROPER CODE SPECIFICATIONS FOR THE SEISMIC DESIGN OF BUILDINGS

M. E. Rodriguez<sup>(1)</sup>

<sup>(1)</sup> Professor, Instituto de Ingenieria, Universidad Nacional Autonoma de Mexico, mrod@unam.mx

### Abstract

Recorded ground motions in several earthquakes in different urban areas around the world exceeded the design basis earthquake considered by building codes in such areas. This has been for example the cases of the 1985 Mexico City earthquake, 2010 Chile earthquake and the 2010 and 2011 Christchurch earthquakes in New Zealand. These earthquakes in Mexico, Chile and New Zealand are studied in this research. Results show the need of revising existing current seismic design procedures specified by building codes. Nonlinear time-history analyses were performed using a SDOF system subjected to typical recorded ground accelerations in these earthquakes, and considering typical structural features of buildings in these earthquakes. Demands of lateral strengths and roof drift ratios in these buildings were obtained from such analyses. In addition, a seismic damage index previously proposed by the author was computed for the selected earthquakes. Results of the evaluation of this damage index showed the importance of properly considering the type of soil and proper amplification of ground movements due to the type of soils when specifying seismic design strength and displacement demands for the seismic design of buildings. These results also showed the importance of specifying both proper drift demands and a type of structural system capable of controlling building displacements during earthquakes. This research reached the simple conclusion that seismic design codes should specify a clear and explicit seismic design procedure using proper values of demand and capacity of inelastic displacements in a building for the design earthquake. Such procedure calls for a displacement-based design as a first step in the seismic design of a building.

Keywords: earthquake damage, drift demands, seismic codes, wall buildings, frame buildings.



### 1. Introduction

Recorded ground motions in several earthquakes in different urban areas around the world exceeded the design basis earthquake considered by building codes in such areas. This has been for example the cases of the 1985 Mexico City earthquake, 2010 Chile earthquake, and the 2010 and 2011 Christchurch earthquakes in New Zealand. These events are studied in this research.

Nonlinear time history analyses were performed using an equivalent SDOF system, representing a multistory building. This SDOF system was analyzed with typical ground accelerations recorded in these earthquakes, and considering typical structural features of buildings. Both lateral strength demands and roof drift ratios in these buildings were obtained from such analyses. In addition, a seismic damage index previously proposed by the author was computed for the selected earthquakes. This was performed considering representative structural features of buildings that experienced the selected earthquakes. Results from the nonlinear dynamic analysis and computed seismic damage index in these earthquakes showed the importance of properly considering the type of soil and proper amplification of ground movements due to the type of soils when specifying design strength and displacements for the seismic design of buildings. In addition, these results showed the importance of both specifying proper drift demands and a type of structural system capable of controlling building displacements during earthquakes. This research reached the simple conclusion that seismic design codes should specify a clear and explicit seismic design procedure using proper values of demand and capacity of inelastic displacements in a building for the design earthquake. Such procedure calls for a displacement-based design as a first step in the seismic design of a building.

### 2. Defining seismic hazard and seismic risk

Seismic risk can be expressed qualitatively as:

Seismic hazard is assessed from Earth sciences. For example, a PGA hazard curve can be obtained using the annual exceedance probability (PE) for a given ground motion from a single characteristic source, v(a), and is defined as (Wang, 2015):

$$p(a) = PE_E PE_a \tag{2}$$

where  $PE_E$  is the annual *PE* for the earthquake, and *PEa* is the *PE* for ground motion. The value of  $PE_E$  depends on the location, magnitude and the recurrence interval of the earthquake, which are highly uncertain, and they are quantified by deterministically or probability models. The probability *PEa* depends on the PGA uncertainty, and is expressed for example using a lognormal distribution, assuming values for the median PGA and standard deviation (Wang, 2015). It follows that seismic hazard is highly probabilistic, and probability models have to be introduced. However, these models can be bad or even wrong (Wang, 2015), which add more uncertainty in seismic hazard assessment. This is the case for example of the 2010 and 2011 Christchurch earthquakes, which in some period ranges significantly exceeded in less than one year both the 500 and 2500-year return period spectra. This made some researchers suggesting the application of time-varying seismic hazard models for a revision of the seismic design spectra for the Christchurch region (Gerstenberger et al., 2014). As shown in the following, additional examples of underestimation of seismic hazard are the cases of the 1985 Mexico City and 2010 Chile earthquakes.



# 3. Comparison of specified and observed seismic demands in the 1985 Mexico City, 2010 Chile, and the 2010 and 2011 Christchurch earthquakes.

Table 1 provides a summary of some ground motions resulting from the 1985 Mexico City earthquake, 2010 Chile earthquake, and the 2010 and 2011 Christchurch earthquakes. Strong motion records are identified in Table 1 with the code CCC for the 2010 and 2011 Christchurch earthquakes. These records were obtained in a station located in the central area of Christchurch, where the highest building damage was observed. For the 2010 Chile earthquake, strong motions recorded in the Concepcion Centro station, in the central area of the city of Concepcion, were chosen. These records were CON\_L (longitudinal) and CON\_T (transverse), see Table1. This station was located in the area of highest building damage observed in this earthquake. Table 1 also shows for the Concepcion Centro station, the value of  $V_{s30}$ , which is the shear wave velocity of soil in top of 30m. The strong motion record chosen for the 1985 Mexico City earthquake was the SCT record, see Table 1, and was obtained in a station located in the area of highest building damage observed in this earthquake.

Table 1 – Strong motion records from the 1985 Mexico City, 2010 Chile, and the 2010 and 2011 Christchurch earthquakes

Station	Code	Date	Magnitude (Mw)	Direction	PGA (g)	PGV (m/s)	Site class ASCE 7-10	V <sub>s30</sub> (m/s)
Christchurch Cathedral College	CCC	4 Sept 2010	7.1	N64E, N26W	0.22	0.54	D <sup>1</sup>	-
	CCC	22 Feb 2011	6.2	N64E, N26W	0.43	0.56	D <sup>1</sup>	-
Concepcion Centro	CON	27 Feb 2010	8.8	L, T	0.39	0.67	D	230
SCT	SCT	19 Sept 1985	8.1 ( <i>M</i> s)	NS, EW	0.17	0.61	Soft soil	-

<sup>1</sup> Clasification of NZLS (2004)

Fig. 1 shows the horizontal spectral elastic acceleration for Christchurch Cathedral College (code CCC in Table 1) from 4 September 2010 and 22 February 2011 events, assuming for  $\xi$ , fraction of critical damping, the value 5%. These spectra are compared with NZS 1170.5 (2004) elastic design spectra for Christchurch for the 500-year and 2500-year return period specified in this Standard. As seen in Fig.1, spectral demands for the second earthquake are higher than those from the earlier one in the period range less than 1.5 s, and for longer periods these two events are similar. Fig.1 also shows that in some period ranges the two events exceeded in less than one year both the 500 and 2500 year return period spectra.



Fig. 1 – Horizontal spectral acceleration ( $\xi = 5\%$ ) for Christchurch Cathedral College from September 4 and February 22 earthquakes compared with the elastic design spectra of NZS 1170. 5 (2004).

Fig. 2 shows the horizontal spectral elastic acceleration for the CON\_L and CON\_T records from the 27 February 2010 earthquake in Chile, assuming  $\xi = 5\%$ , and they are compared with the elastic design spectra specified by the Building Standard in Chile at that time, NCh 433 (1996). As seen in Fig 2, in the period range of about 1.5s to 2.5 s, the event exceeded the elastic design spectra.



Fig. 2 - Horizontal spectral acceleration ( $\xi = 5\%$ ) for Concepcion Centro from 2010 February 22 earthquake compared with the elastic design spectra of NCh 433 (1996).

Fig. 3 shows the horizontal spectral elastic acceleration for the SCT record from the 19 September 1985 event, assuming  $\xi = 5\%$ , and they are compared with the elastic design spectra specified by the Mexico City Building Code at that time (RCDF, 1976). As seen in Fig.3, in the period range of about 1.5s to 2.8s, the event exceeded the elastic design spectra.



Fig. 3 - Horizontal spectral acceleration ( $\xi = 5\%$ ) for SCT station from 1985 September 19 earthquake compared with the elastic design spectra of RCDF (1976).

A common feature observed in the three earthquakes is that in all of them, in some period ranges the observed seismic demands (expressed as acceleration spectra) were higher than those specified by the corresponding local building codes. In some cases the observed demands exceeded the specified values in about 100%. This shows the inherent uncertainties of estimating seismic hazard. It also shows that because code designed buildings will reach their lateral strength capacity in the design earthquake, building codes in seismic areas ought to emphasize not only on providing enough lateral strength in a structure, but also in designing buildings with a seismic resistance system capable of controlling lateral displacement demands in a strong earthquake. This is elaborated in the following.

## 4. Approximated analysis of the inelastic response of buildings in the 1985 Mexico and 2010 Chile earthquakes

To study the inelastic response of buildings in the 1985 Mexico and 2010 Chile earthquakes, a simple approach to estimation of global lateral displacements of multistory buildings was selected. This approach uses an SDOF system to compute these displacements. The response of this system is also used later for computing a proposed damage index for the selected earthquakes.

4.1 Approximate lateral displacement analysis of buildings subjected to strong earthquakes

A simple procedure is used in this study for estimation of lateral displacements in buildings subjected to strong earthquakes. This procedure is described in detail in Rodriguez (2015), and only a brief description of such procedure is given in the following.

A constant deflected shape is assumed for the seismic analysis of multistory buildings, and the roof displacement  $\delta$  is selected as the response parameter of an equivalent SDOF system (Saiidi and Sozen, 1981).

The maximum roof drift ratio in a multistory building,  $D_{rm}$ , is defined as follows:

$$D_{rm} = \frac{\delta_m}{H} \tag{3}$$

where  $\delta_m$  is the maximum roof displacement and H is the height of the building above ground level.

For a regular building with n floors and a constant story height, h, the following expression can be written:



$$H = nh \tag{4}$$

In addition, the fundamental period of a building, T, can be related to the number of floors, n, by the following expression:

$$T = \frac{n}{\lambda} \tag{5}$$

The analysis procedure used in this study assumes an effective fundamental period equal to  $\sqrt{2}$  times the value for the uncracked section (Shimazaki and Sozen, 1984). An amplification of the fundamental period of a building due to soil-structure interaction was also considered for the analysis of the seismic response of buildings in the soft-soil area of Mexico City (Rodriguez, 2015). With these assumptions, and considering only frame buildings for Mexico City, a value of 5.4 s<sup>-1</sup> for  $\lambda$  was selected for the analysis of the 1985 Mexico Earthquake, and for the 2010 Chile Earthquake only wall buildings were considered, with the value of 14 s<sup>-1</sup> for  $\lambda$ .

In this study, the seismic response of a building is related to the response of a SDOF system. A basic assumption of this procedure is that the fundamental circular frequency,  $\omega$ , and the maximum global displacement ductility ratio,  $\mu_{,}$  of a multistory building, are equal to the circular frequency and maximum displacement ductility ratio, respectively, of an equivalent SDOF system. With these assumptions, and considering results from basic modal analysis, the roof displacement,  $\delta_m$ , and maximum lateral displacement of the SDOF system,  $S_d$ , can be related by means of the first-modal participation factor,  $\Gamma_1$ , and contribution factor  $\Gamma_1^*$ , using the following expression:

$$\Gamma_1^* = \frac{\delta_m}{S_d} \qquad \text{and} \qquad \Gamma_1^* = \Gamma_1 \phi_{1,n} \tag{6}$$

where  $\phi_{1,n}$  is the first-modal shape at the uppermost level of the building, *n*. Combining Eqs. (3) through (6), Eq. (3) can be expressed as

$$D_{rm} = \frac{\Gamma_1^* S_d}{T \lambda h} \tag{7}$$

Another form of Eq. (7) can be obtained from Eqs. (3) and (6):

$$D_{rm} = \frac{\Gamma_1^* S_d}{T\left(\frac{H}{T}\right)} \tag{8}$$

From Eqs. (7) and (8), it follows that:

$$H/T = \lambda h \tag{9}$$

It is of interest that values for the ratio  $H/T_o$  for typical buildings in Chile constructed in the period 1940-2010 have been obtained by Massone et al. (2012), and Lagos et al. (2012), where  $T_o$  is the fundamental period of a building assuming gross section properties for all elements. According to these authors, most buildings in Chile have  $H/T_o$  ratios between 40 and 140 m/s, with a mean value of about 70 m/s. Values above 40 m/s represent typical normal stiffness buildings, and values over 70 m/s correspond to stiff buildings in Chile . Values of the H/T ratios for these buildings can be obtained considering the ratio  $T/T_o$  equal to  $\sqrt{2}$ . Since according to Eq. (9), H/T is directly proportional to  $\lambda$ , it follows that high values of  $\lambda$  or low H/T ratios pertain to flexible buildings. As can be seen in Eqs. (7) or (8), these properties of  $\lambda$  or H/T are relevant when computing lateral displacements in buildings subjected to strong earthquakes, and this will be shown in the following.

Spectral displacement  $S_d$  was computed using the Ruaumoko computer program (Carr, 2011) for given ductility displacement ratios,  $\mu$ , equal to 1, 2 and 4. The Modified Takeda Hysteresis rule (Carr, 2011) was selected for this study, with the unloading stiffness factor equal to 0.4, and the reloading stiffness factor equal to



0.6. For consistency with implicit values of fraction of critical damping,  $\xi$ , assumed by building codes, the assumed value for  $\xi$  was equal to 5%. However, it must be noted that Panagiotou (2008), and Martinelli and Filippou (2009), suggest that time-history nonlinear analysis using such damping value may underestimate the probable lateral displacement of buildings responding to strong earthquakes.

Roof drift ratios  $D_{rm}$  for typical buildings responding to the 1985 Mexico record and 2010 Chile record (CON\_L record) were computed using Eq. (7) and the corresponding values of  $\lambda$  above commented for these buildings. In addition, computed values of  $S_d$  following the above described procedure, and the values of 5/4 and 3m, for  $\Gamma_1^*$  (Sozen, 1997) and *h*, respectively, were also used in these calculations. Results for computed values of  $D_{rm}$  are shown in Figs. 4 and 5, for typical buildings that experienced the 1985 Mexico Earthquake and 2010 Chile Earthquake, respectively. The term DUC in these figures stands for the ductility displacement ratio,  $\mu$ .



Fig.  $4 - D_{rm}$  spectra for SCT record from 1985 Mexico earthquake



Fig. 5 –  $D_{rm}$  spectra for Concepcion Centro record from 2010 Chile earthquake

A first observation on results in Figs 4 and 5 is that due to the inherent higher flexibility of frame buildings as compared to that of stiff buildings, computed values of roof drift ratios in frame buildings for the 1985 Mexico record are significantly higher than those computed for stiff buildings in Concepcion in the 2010 Chile earthquake, with differences of the order of 100%. These high values of probable roof drift ratios in buildings in the 1985 Mexico earthquake explains the important structural damage or even collapse of buildings



observed in that earthquake, especially in frame buildings that did not satisfy detailing provisions of modern codes. A second observation on results in Figs 4 and 5 is that values of computed roof drift ratio demands in wall buildings responding to the 2010 Chile record were lower than 0.01, which is considered a lower limit of the displacement capacity of slender walls without confined boundary elements (Wood, 1991). This observation is consistent with the relative low rate of collapses of buildings in Concepcion in the 2010 Chile earthquake. In this earthquake only a 15-story wall building collapsed. A third observation on results in Figs 4 and 5 is that for period ranges higher than about 1.7 s and 1.3 s, for the 1985 Mexico City record and the 2010 Chile record, respectively, less drift ratios would be expected when the inelastic response increases. However, for period ranges smaller than the later values, when the inelastic response increases, more drift ratios would be expected. Considering some similarities on the type of site soil properties corresponding to these records, later discussed, the third observation suggests that cases of soft soil lead to values of inelastic spectral displacements that do not follow the equal-displacement rule, typically assumed for cases of different site soil classification, such as for example rock. These observations are also consistent with computed values of a proposed damage index for buildings responding to earthquakes, which are shown in the following.

#### 4.2 Evaluation of a damage index for buildings in the 1985 Mexico and 2010 Chile Earthquakes

A damage index,  $I_d$ , has been proposed by Rodriguez (2015), and is defined as:

$$I_d = \frac{\Gamma_1^{*2} E_H}{\left(\omega H D_{re}\right)^2} \tag{10}$$

where  $E_H$  is the hysteretic energy per unit mass dissipated by the same equivalent SDOF system that was analyzed for obtaining  $D_{rm}$ ,  $\omega$  is the natural circular frequency of the system, and  $D_{rc}$  is the maximum roof displacement in a building responding in the linear range and absorbing at collapse of the building an energy equal to  $\Gamma_1^{*2} E_H$ . The value of 0.025 for parameter  $D_{rc}$  was obtained from the best fit between computed values of  $I_d$  and the observed global building damage for a set of worldwide earthquakes (Rodriguez, 2015).

The product  $\omega$  *H* in Eq. (10) when expressed as  $2\pi$  *H*/*T* can be related to either *H*/*T* or  $\lambda h$ , see Eq. (9). When using these relationships, Eq. (10) can be expressed as:

$$I_{d} = \frac{\Gamma_{1}^{*2} E_{H}}{\left(2\pi \frac{H}{T} D_{rc}\right)^{2}}$$
(11)

While the roof drift ratio  $D_{rm}$  is inversely proportional to the ratio H/T, see Eq. (8), Eq. (11) shows that the damage index  $I_d$  is inversely proportional to the square of the ratio H/T, which shows the importance of this ratio for controlling damage in buildings responding to strong earthquakes.

In the evaluation of  $I_d$  for the selected earthquakes, the values assumed for parameter  $\Gamma_1^*$  and ratio H/T were equal to those assumed in the above described evaluation of  $D_{rm}$ . When using these values in Eq. (11) we obtain:

$$I_d = 0.243 E_H$$
 (12)

for frame buildings in the 1985 Mexico Earthquake

and

$$I_d = 0.036 E_H$$
(13)

for wall buildings in the 2010 Chile Earthquake in Concepcion.

Results of the evaluation  $I_d$  for the respective earthquakes were obtained using Eqs (12) and (13) for  $\mu$  values equal to 2 and 4. These results are shown Fig. 6. These plots suggest, in the period range of about 1.5 to 2.5 s, potential of collapse for frame buildings responding to the 1985 Mexico record, whereas for wall buildings



responding to the 2010 Chile record there is no indication of potential of collapse for these buildings. These results are consistent with observed behavior of buildings in the selected earthquakes.



Fig. 6 – Damage index for SCT record from 1985 Mexico earthquake and for Concepcion Centro record from 2010 Chile earthquake

It is of interest that the shape of the damage spectra for the 2010 Chile record has some similarities to the damage spectra of the 1985 Mexico record, see Fig. 6. For both records, for a period range higher than the predominant period of the ground motion,  $T_g$ , which corresponds to the highest demand of  $E_H$ , less building damage is expected when the level of inelastic response increases. However, for the period range smaller than the value of  $T_g$ , when the inelastic response increases, more building damage is expected. Although according to the ASCE7-10 site classification, the area of the city of Concepción, where the 2010 Chile ground motion was recorded, is classified as Site Class D, stiff soil, the above-mentioned similarities between the damage spectra for the 1985 Mexico City and 2010 Chile records suggest that a type of soft soil is present in the Concepcion area. This suggests the importance of properly considering soil properties in the earthquake-resistant design of buildings.

### 5. Conclusions

1. A comparison of specified and observed seismic demands in the 1985 Mexico, 2010 Chile, and 2010 and 2011 Christchurch earthquakes indicates that seismic demands experienced in these three events exceeded the demands specified by local codes, with differences of about 100% for some period ranges. This shows the inherent uncertainties of estimating seismic hazard. It also shows that because code designed buildings will reach their lateral strength capacity for the design basis earthquake, building codes in seismic areas ought to emphasize not only on providing enough lateral strength in a structure, but also in designing buildings with a seismic resistance system capable of controlling lateral displacement demands in a strong earthquake.

2. The low lateral stiffness of frame systems, as compared to that of wall building, lead to values of computed damage index for these buildings that were significantly higher than those found for wall buildings. This is consistent with the observed high rate of damage or collapses in frame buildings in Mexico City during the 1985 Mexico earthquake, as compared with the rate of damage or collapses observed in buildings in Concepcion in the 2010 Chile earthquake.

3. The high values of computed roof drift ratios in buildings for the 1985 Mexico record are consistent with the important structural damage and collapses in frame buildings in the 1985 Mexico City earthquake, especially in those buildings that not satisfied detailing provisions of modern codes. It was also found that values of computed roof drift ratio demands in wall buildings for the 2010 Chile record were lower than 0.01, which is



considered a lower limit of the displacement capacity of slender walls without confined boundary elements. This is consistent with the relative low rate of collapses of buildings in the 2010 Chile earthquake.

4. Results found in this study show that wall buildings should be favored as a desirable structural system in multistory buildings in seismic areas. However, as observed in medium-rise RC wall buildings that collapsed or had severe structural damage in the 2010 Chile earthquake, unless wall buildings are provided with a good number of robust walls, special detailing is needed in RC walls to resist strong earthquakes.

5. Seismic design codes should specify not only the required lateral strength of buildings for the design basis earthquake, but also a clear and explicit seismic design procedure using proper values of demand and capacity of inelastic displacements in a building for the design basis earthquake. Such procedure calls for a displacement-based design as a first step in the seismic design of a building.

### 6. Acknowledgements

The financial support of CONACYT, Contract No 167445, is gratefully acknowledged. Thanks are due to Professor M. Sozen, from Purdue University, and to Professor J. Restrepo, from UC San Diego, for their useful comments.

### 7. References

- [1] Wang Z. (2015) "Predicting or Forecasting Earthquakes and the Resulting Ground-Motions Hazards: A Dilemma for Earth Scientists", Seismological Research Letters, **86** (8), 1-5.
- [2] Gerstenberger M., McVerry G., Rhoades D. and Stirling M. (2014), "Seismic Hazard Modeling for the Recovery of Christchurch", Earthquake Spectra, **30** (1), 17-29.
- [3] Standards New Zealand (2004), NZS 1170.5:2004, "New Zealand Standard. Structural design actions. Part 5: Earthquake actions-New Zealand".
- [4] Instituto Nacional de Normalizacion (1996), "Norma Chilena Oficial. Diseño Sísmico de edificios". NCh433.Of 1996. Chile
- [5] RCDF (1976), "Reglamento de Construcciones para el DF". (Building Code for Mexico City).
- [6] Rodriguez M. E. (2015) "Evaluation of a proposed damage index for a set of earthquakes", Earthquake Engineering & Structural Dynamics, 44, 1255-1270.
- [7] Saiidi M. and Sozen M. (1981), "Simple nonlinear analysis of RC structures", Journal of Structural Engineering, ASCE, **197**, 937-952.
- [8] Shimazaki, K. and Sozen, M. (1984), "Seismic Drift of Reinforced Concrete Structures", Technical Report, Hazama-Gumi, Tokyo, 1984, 145-165
- [9] Massone L., Bonelli P., Lagos R., Luders C., Moehle J. and Wallace J. (2012) "Seismic Design and Construction Practices for RC Structural Wall Buildings", Earthquake Spectra, **28** (S1), S245-S256
- [10] Lagos R., Kupper M., Lindenberg J., Bonelli P., Saragoni R., Gueldelman T., Massone L., Boroschek R., and Yanez F. (2012), "Seismic Performance of High-Rise Concrete Buildings in Chile", International Journal of High-Rise Buildings, 1 (3), 181-194.
- [11] Carr, A. (2011), "RUAUMOKO, Computer Program Library", University of Canterbury, Department of Civil Engineering
- [12] Panagiotou, M. (2008), "Seismic Design, Testing, and Analysis of Reinforced Concrete Wall Buildings", Doctoral Thesis, University of California, San Diego, USA, supervised by J. Restrepo.
- [13] Martinelli, P. and Filippou, F. (2009), "Simulation of the shaking table test of a seven-story shear wall building", Earthquake Engineering & Structural Dynamics, **38**, 587-607.
- [14] Wood, S. (1991), "Performance of Reinforced Concrete Buildings During the 1985 Chile Earthquake: Implications for the Design of Structural Walls", Earthquake Spectra, **7** (4), 607-638.