

The 2010 Maule, 2011 Christchurch and 2011 Great East Japan Earthquakes. Lessons Learned for the Seismic Design of Structures

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SUMMARY: (10 pt)

Some ground motions recorded in the 2010 Maule, the 2011 Great East Japan, the 2011 Christchurch and the 1985 Mexico Earthquakes and simple analytical models are used in this paper for analyzing the structural response of typical buildings during these earthquakes. Lateral displacements of structures, hysteretic energy response spectra and a measure of earthquake damage are calculated in this paper using the selected records. From results found in the analysis conducted in this paper, seismic design implications for structures are discussed. It is found that structures on soft soil need to have special seismic design considerations.

Keywords: Damage parameter, seismic design, 2010 Chile Earthquake, 2011 Great East Japan earthquake, 2011 Christchurch Earthquake

1. INTRODUCTION

The recent 2010 Chile and 2011 Great East Japan earthquakes had high magnitudes, 8.8 and 9.0, respectively. It is relevant to evaluate the observed response of structures during these strong earthquakes along with results from response spectra analysis using recorded ground motions. In this paper, results from response spectra analysis and a measure of seismic damage corresponding to the 2010 Chile, the 2011 Christchurch and 2011 Great East Japan earthquakes are calculated. As a comparison, these parameters are also evaluated for the 1985 Mexico City earthquake. Results from the evaluation of these parameters are used to discuss some seismic design implications for structures.

2. EARTHQUAKE GROUND MOTIONS AND OBSERVED BUILDING BEHAVIOR

In this study four accelerograms recorded in four earthquakes are analyzed. These earthquakes in chronological order are 1985 Mexico City, 2010 Chile, 2011 Christchurch and 2011 Japan. Table 1 shows typical characteristics of the earthquake ground motions that have been used, which include Magnitude M_s , epicentral distance, soil type at the recording site, peak ground acceleration, PGA (g), where g is the acceleration of gravity, peak ground velocity, PGV (cm/s), and the abbreviations used for the records selected. A detailed description of the strong ground motions observed in the 2011 Christchurch Earthquake including the ground motion record selected for this study is given by Bradley and Cubrinovski (2011).

Building behavior observed during these earthquakes is documented in the literature. According to a review of this information, the most destructive earthquakes were the ones experienced in Mexico City (1985) and Chile (2010).

Table 2.1. Earthquake Data

No	Earthq.	Year	Station	Comp.	Abbr.	Site* Class	Epic. Dist. (km)	M _s (M _w)	PGA (g)	PGV (cm/s)
1	Chile	2010	Concepcion	NS	CON-1	D	65	8.8	0.40	68
2	Japan	2011	Sendai	NS	MYG013	C	126.1	9.0	1.55	75
3	Christchurch	2011	N64E	EW	N64E	D	6	6.3	0.48	71.4
4	Mexico	1985	SCT	EW	SCT	Soft soil	400	8.1	0.17	61

*ASCE 7-05 Site Classification: C ($V_s= 360$ to 760 m/s), D ($V_s= 180$ to 360 m/s)

3. APPROXIMATE LATERAL DISPLACEMENT ANALYSIS OF STRUCTURES

A simple approach is chosen here for the evaluation of lateral displacements, which uses SDOF systems to analyze global displacements of multistory buildings. A constant deflected shape is assumed for the seismic analysis of multistory buildings and the roof displacement, δ , is selected as response parameter of an equivalent SDOF system (Saïidi and Sozen, 1981).

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

$$D_{rm} = \frac{\delta}{H} \quad (3.1)$$

where δ_m is the maximum roof displacement and H is the height of the building.

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

$$H = nh \quad (3.2)$$

In addition, the fundamental period of a building, T , and n can be related by

$$T = \frac{n}{\lambda} \quad (3.3)$$

The parameter λ depends of the type of structure. For structural wall buildings on firm soil as those designed according to the Chilean and Japanese practice, results from small amplitude ambient vibration tests suggest a value of λ equal to 20 (Rodriguez and Aristizabal, 1999). For typical frame buildings on firm soil constructed before 1985 in Mexico City a value of 10 has been suggested for λ (Rodriguez and Aristizabal, 1999). Lower values of λ should be used for buildings in soft soil as that of the Mexico City case. A value of $1.3T$ has been suggested for evaluating the SSI period, where T is evaluated considering the fixed-base case (Rodriguez and Aristizabal, 1999). A reduction of lateral stiffness should be considered when analyzing the response of buildings during earthquakes. As an approximate procedure, it is assumed in this study that the effective fundamental period of a building is equal to $\sqrt{2}$ times the fundamental period of vibration obtained from small amplitude vibration tests. Thus, when evaluating D_{rm} , the previous discussed values should be affected by the factor $\sqrt{2}$. According to this discussion, a value of λ equal to 14 it is assumed in this paper when analyzing buildings in Japan. For the Chile Earthquake, considering that the analyzed ground motion was

obtained in a type of soft soil, a value of λ equal to $14/1.3=10.8$ is used. For Mexico City, considering flexible frame buildings on soft soil, a value of λ equal to $7/1.3=5.4$ is used. For the Christchurch Earthquake, considering a type of frame buildings and that the analyzed ground motion was obtained in a type of soft soil, a value of λ equal to $10/1.3=7.7$ is used.

The seismic response of a building is related here to the response of a SDOF system with a lateral displacement u and yielding displacement u_y . A basic assumption of this procedure is that the fundamental circular frequency, ω , and the maximum global displacement ductility ratio μ_m , in the multistory building are equal to the circular frequency and maximum displacement ductility ratio of the SDOF system, respectively. With these assumptions, δ_m and u_y can be related by means of the parameter Γ using the following expression:

$$\Gamma = \frac{\delta_m}{\mu_m u_y} \quad (3.4)$$

when performing a global seismic analysis of regular building structures, parameter Γ can be approximated to $5/4$ (Sozen, 1997).

Combining Eqns. (3.1)-(3.4), Eqn. (3.1) can be written as:

$$D_{rm} = \frac{\mu_m \Gamma u_y}{T \lambda h} \quad (3.5)$$

Displacement u of the SDOF system was computed for given ductility displacement ratios, μ (1,2,4) and considering a fraction of critical damping, ξ , equal to 0.02, using the RUAUMOKO computer program (Carr, 2011) and the Takeda hysteresis rule, with a stiffness, k_u , defined as:

$$k_u = k_o \mu^{-0.5} \quad (3.6)$$

where k_o is the initial stiffness of the system. The reloading stiffness parameter was taken equal to 0.5.

Fig. 3.1 shows plots of D_{rm} spectra using the above discussed values for the parameters involved in Eqn. (3.5) and considering $h = 3\text{m}$. An inspection of inelastic results shows that in general the highest roof drift ratios correspond to frame systems responding to the SCT Mexico City record, with the highest demands in the period range of about 1.2-2.5 s, that is buildings in the range of 7-14 stories. Results found using the Concepcion record (CON-1), recorded in the area of highest building damage in the city of Concepcion, show that the highest roof drift ratios correspond to wall buildings in the period range of about 0.9-2.2 s, see Fig. 3.1, that is buildings in the range of 10-24 stories. In both cases, Mexico City and Concepcion, the highest rate of building damage was observed in the above mentioned range of number of stories.

For the 2011 Japan earthquake, results using the Sendai record indicate maximum values of D_{rm} of about 0.01 in the range of 0.5-1.0 s, that is in the range of 7-14 stories. For the Christchurch record, maximum values of D_{rm} , from 0.01 to 0.02, were calculated in the range of 0.5-1.7 s, that is in the range of 4-13 stories.

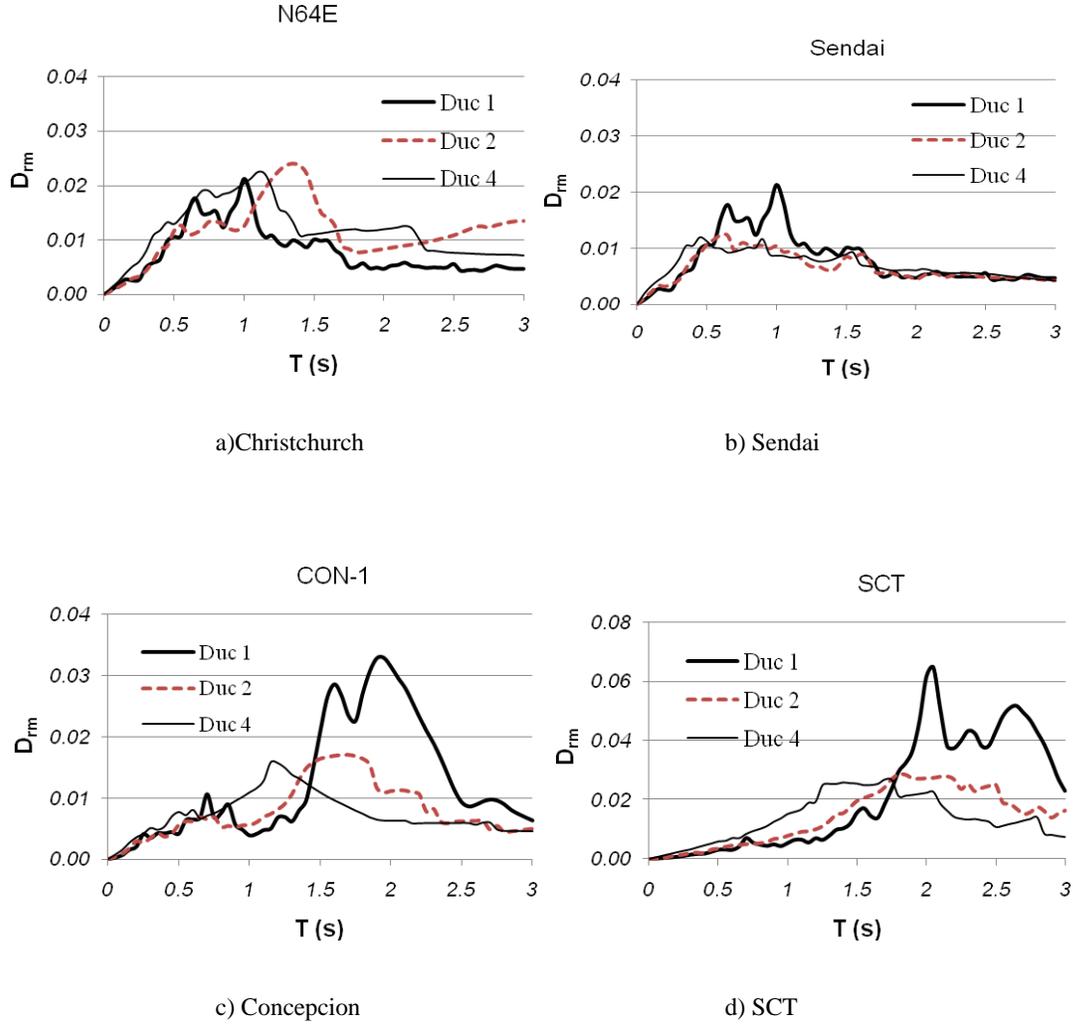


Figure 3.1. D_{fm} spectra for 4 earthquake ground motion records

4. SEISMIC RESISTANCE

By specifying a displacement ductility ratio, μ , for an inelastic SDOF system for a given earthquake record, the seismic resistance, C_y , is defined as:

$$C_y = \frac{R_y}{M g} \quad (4.1)$$

where R_y is the yielding resistance and M is the mass of the structure.

Fig. 4.1 shows results of the evaluation of C_y for the Christchurch, Sendai and Concepcion records, which were calculated using elastic and inelastic analysis of the same SDOF system above described. As shown in Fig. 4.1, the maximum demand of inelastic seismic resistance for the Christchurch record

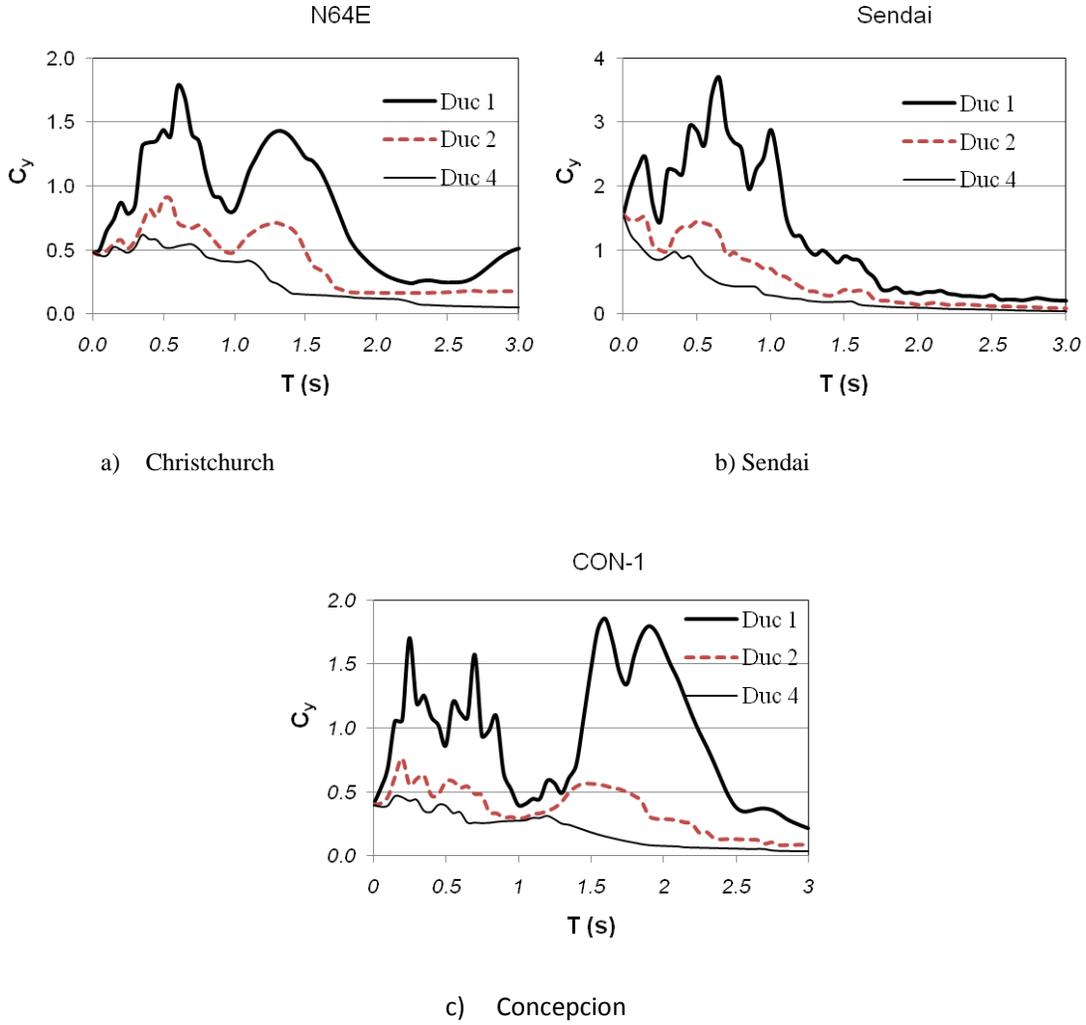


Figure 4.1 C_y spectra for 3 earthquake ground motion records

ranged from 0.5g to about 0.9g in the period range 0-1.5 s. The maximum demand of inelastic seismic resistance for the Sendai record led to values of about 1.5g to 0.5g in the period range 0-1.1 s. Figure 4.1 shows C_y demands for the Concepcion record. As seen there, maximum inelastic demands of C_y ranged from about 0.5g to 0.7g in the range 0.15 to 0.7s.

5. PARAMETER FOR MEASURING SEISMIC DAMAGE

For a MDOF structure with fundamental circular frequency ω and height H , the parameter I_d (Rodriguez, 1994) for measuring seismic damage is defined as:

$$I_d = \frac{\Gamma^2 E_H}{(\omega H D_{rd})^2} \quad (5.1)$$

where E_H is the hysteretic energy per unit mass dissipated by the SDOF system representing the MDOF system, and D_{rd} is the maximum roof drift ratio in a building associated to an acceptable building performance in a strong earthquake. In this paper it is assumed the value of 0.015 for parameter D_{rd} , for both wall and frame buildings.

A parametric form of the product ωH for regular buildings has been proposed by Rodriguez (1994):

$$\omega H = 2\pi \lambda h \quad (5.2)$$

Figure 5.1 shows results of the evaluation of I_d for the cases of μ equal to 2 and 4 for the selected earthquake ground motions. The assumed values for λ and h for evaluating I_d were those used for the previous discussed evaluation of D_{rm} . Results shown in Fig. 5.1 indicate that the highest damage parameter correspond to the SCT record (Mexico City, 1985). However, results using the Concepcion record indicate also high values of the damage parameter, particularly in the range of 1-1.8 s, that is in the range of about 10 to 20 stories. It is of interest that the peak value of I_d for the SCT record is only about 50% higher than the peak value corresponding to the Concepcion record. Furthermore, the shape of the I_d spectra for the Concepcion record has some similarities to that of the demands of I_d for the SCT record, see Fig. 5.1, which suggests a type of soft soil in the areas where the mentioned accelerograms were recorded. The above discussed computed values of I_d and shape of the I_d spectra for the SCT and Concepcion records explain the important building damage or cases of failures observed in the 1985 Mexico and 2010 Chile earthquakes in Mexico City and Concepcion, respectively.

For the Christchurch record, maximum values of parameter I_d were comparable to the demands corresponding to the Sendai record (Fig. 5.1), although slightly higher in the period range equal to 1.1-1.5s. Maximum I_d values for the case of the Sendai record are equal to about 1.0, see Fig. 5.1, which is a value much lower than the maximum values for I_d obtained using the SCT and Concepcion records. These findings suggest for the Sendai and Christchurch cases expected building damage much lower than that observed in the 1985 Mexico and 2010 Chile earthquakes.

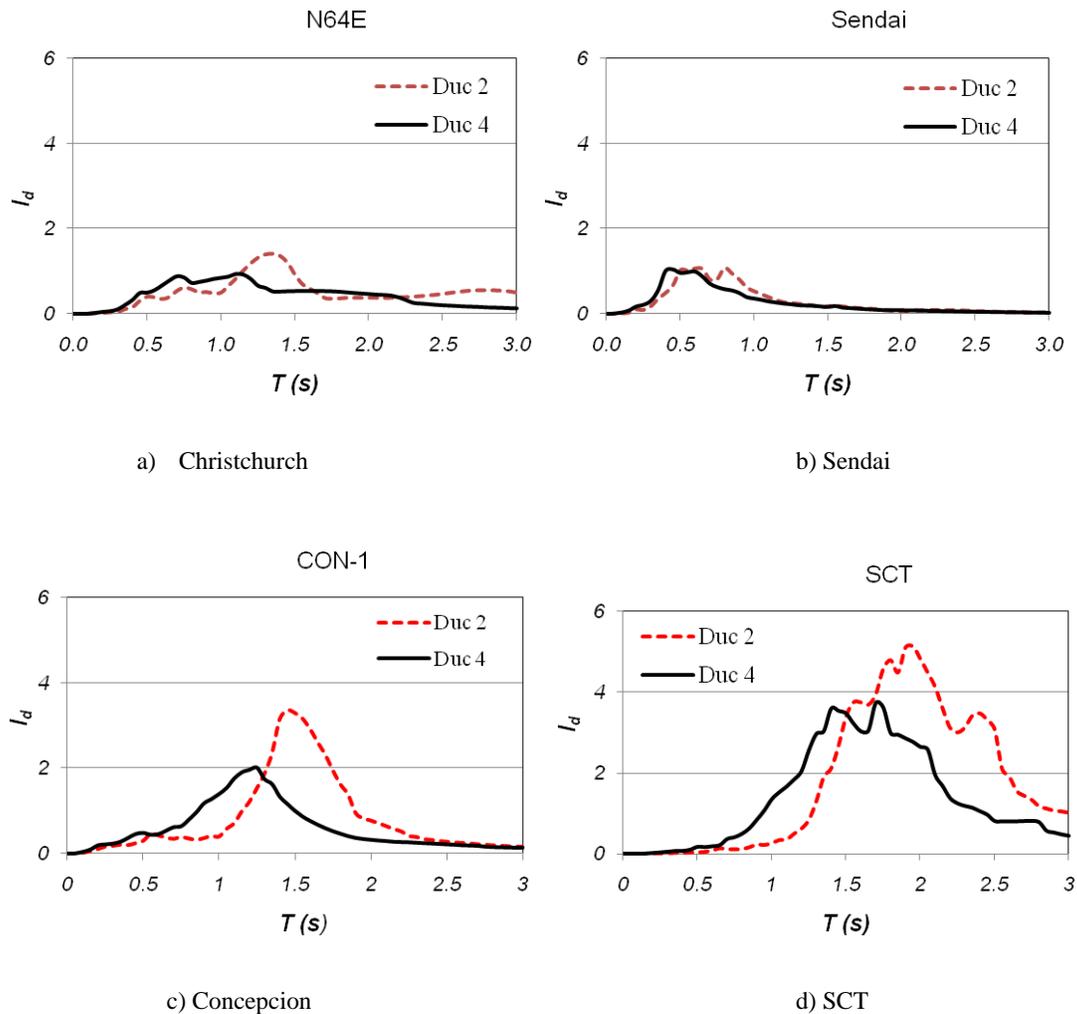


Figure. 5.1 Measure of seismic damage for 4 earthquake ground motion records

6. CONCLUSIONS

A seismic damage parameter formerly proposed by the author is used in this study to evaluate seismic damage using data from the recent 2010 Chile, 2011 Christchurch and 2011 Great East Japan earthquakes. For comparison with results from this evaluation, seismic damage is also evaluate for the case of 1985 Mexico City earthquake.

Results from the evaluation show an acceptable correlation with global building damage observed in the earthquakes studied. It is also found that is relevant controlling lateral displacements, mainly roof drift ratio of buildings, for minimizing seismic damage.

The 2010 Chile earthquake also showed the importance of properly considering soil properties for the seismic design of buildings since buildings in a type of soft soil in Concepcion had computed values of the proposed seismic damage parameter that were comparable to those corresponding to the Mexico City case not only in maximum values of this parameter but also in the shape of the spectra for the damage parameter. A number of RC structural wall buildings on this type of soil in Concepcion either collapsed or had severe structural damage during the 2010 Chile earthquake.

The computed values of the damage parameter for the case of the analyzed ground motions recorded in the 2011 Christchurch and 2011 Great East Japan earthquakes were in general significantly lower than those corresponding to the 2010 Chile, and 1985 Mexico City earthquakes, which is in agreement with the observed building damage in these earthquakes.

The above conclusions suggest that the proposed damage parameter can be used as a useful and simple tool for an innovative seismic design approach and for evaluating expected seismic performance of existing regular structures.

ACKNOWLEDGMENT

The author acknowledges the assistance of his formerly graduate students for developing the proposed damage index, and the support of the Instituto de Ingenieria, UNAM, Mexico City, to carry out this research.

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