RESPONSE SPECTRA AND BUILDING RESPONSE IN CHILE

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ABSTRACT

The seismic behavior of 45 reinforced concrete buildings between 11 and 23 stories and the response spectra measured in Viña del Mar city for the 1985 chilean earthquake are used to estimate the design spectra in epicentral zone of Chile.

Since most of the response of these buildings was almost elastic. Their good behavior can not be only explained through ductility reductions due to nonlinear incursions. Therefore an additional reduction factor due to earthquake source mechanism between 4 to 5 is proposed to obtain the design spectra.

INTRODUCTION

The March 3, 1985 earthquake is by far the best instrumented major earthquake in South America and is one of the best instrumented major earthquake above 7.5 in the world to date.

The magnitude 7.8 earthquake originated off-shore Valparaiso was recorded by 31 accelerographs of the high density Central Chile accelerograph network. The earthquake was recorded in Llolleo with 0.67 g and in Viña del Mar with 0.36 g in epicentral conditions. (Ref. 1).

The accelerograph network recorded the earthquake in cities with large number of reinforced concrete buildings designed according to U.S.A., and German practice. This situation allows a world opportunity to relate observed damage in the epicenter zone with recorded ground motion.

The response spectra of Llalleo station is the closest to epicenter.

However the most important case correspond to Viña del Mar, where 304 reinforced concrete building between 5 and 23 stories were affected by the earthquake.

In this study the 45 buildings of more than 10 stories of Viña del Mar are considered to estimate the design spectra of the chilean code.

In table 1 a comparison between Viña del Mar and Llolleo is made. Despite the maximum ground acceleration $\ddot{u}_s$ max of Llalleo in the NS direction is 1.88 times greater than Viña del Mar and the

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destructiveness potential factor is 1.58 greater. It is necessary to compare both response spectra in the range of natural period of Viña del Mar high rise buildings.

TABLE 1. Comparison of 1985 Chile Earthquake.
Viña del Mar and Llolleo Station.

Station	Component	$\ddot{u}_{smax}$	$10^{-4}\,P_{D}$ g sec^{3}
Viña del Mar	S 20º W (NS) N 70º W (EW)	0,356 g 0,228 g	117,81 55,75
Llolleo	N 10º E (NS) S 80º E (EW)	0,669 g 0,426 g	196,22 78,40

DYNAMIC CHARACTERISTICS AND SEISMIC BEHAVIOR OF VIÑA DEL MAR BUILDINGS

During the earthquake 304 buildings between 5 and 23 stories were affected by the earthquake in Viña del Mar. (Table 2) For the study only the buildings of more than 10 stories were considered. The number of buildings between 11 and 23 stories are 45.

TABLE 2. Viña del Mar Buildings.
Distribution by Number of Stories.

Number of Stories	Number of Bldg.
5	93
6 - 7	69
8 - 9	70
10 -11	35
12 -15	21
16 -19	6
20 -23	10
Total	304

The 45 buildings to be considered in this study are of reinforced concrete, most of them of shear wall type.

The main axes of these buildings are in NS and EW direction.

The building distribution of the buildings according to the number of stories is given in Table 3.

The damage survey of the buildings was performed. The classification of the damage of the 45 buildings was: 7 buildings without damage, 33 with light damage, 3 buildings with moderate damage and 2 with severe damage (without collapse). The damage classification criteria considered in this study is given in reference 2 in detail. In Table 4 is given the summary of the damage.

38 buildings have light damage (84,4%) with a global ductility less than 2. Most of these building probably behave in the elastic range during the earthquake despite the large ground acceleration recorded.

TABLE 3. Distribution of Viña del Mar Buildings of more than 10 Stories.

Nº Stories	Number	%	∑ %
23	1	2,2	2,2
22	5	11,1	13,3
21	3	6,7	20,0
20	0	0	20,0
19	0	0	20,0
18	1	2,2	22,2
17	2	4,4	26,6
16	1	2,2	28,8
15	3	6,7	35,5
14	4	8,9	44,4
13	4	8,9	53,3
12	9	20,0	73,3
11	12	26,7	100,0
Total	45	100,0	100,0

From all the 45 buildings the natural period after the earthquake was measured using ambient vibrations.

The measured values for each building are given in (Ref. 2). The range of natural period in the NS direction vary between 0,35 and 1,18 sec., and in the EW direction between 0,40 and 1,30 sec.

TABLA 4. Viña del Mar Buildings Damage Level.

Damage Level	Number of Buildings
Undamaged	7
Light Damage	31
Moderated Damage	5
Severe Damage	2

ACCELERATION RESPONSE SPECTRA OF VIÑA DEL MAR AND LLOLLEO

The response spectra obtained in Llolleo corresponds to the maximum epicenter conditions. However the spectra of Viña del Mar corresponds to the highest concentration of reinforced concrete buildings in the epicenter zone. Therefore this former spectra allows to study the cause-effect of the earthquake. The spectra as the cause or seismic requirement and building damage as the effect.

In this study the 5% damping acceleration response spectra are considered. The response spectra of Viña del Mar for both direction were compared with the ones obtained for Llolleo. It was found that values of both spectra are similar in the range between 0,35 to 1,2 seconds of measured natural periods of Viña del Mar buildings. The comparison is shown in Fig. 1. In this figure are indicated with black dot the measured natural periods of the 45 buildings. The Llolleo N 10° E records has a maximum ground acceleration of 0.669 g and the Via del Mar S 20° W is 0.356 g.

The comparison of the spectra of both components shows a strong directivity of the earthquake in the NS direction. Therefore the acceleration response spectra of the S 20° W (NS) component of Viña del Mar represents the extreme seismic requirements for structures with natural periods between 0.40 to 1.20 secs.

A METHOD TO ESTIMATE AVERAGE RESPONSE SPECTRA

The average relative displacement response spectra $E\{S_D(T_{n,\eta})\}$ can be estimated using the method suggested by Crempien and Saragoni for non-stationary model of accelerograms (Ref. 3).

$$E\{S_D(T_n \eta)\} = 0.884 \sqrt{\beta \left[\frac{\gamma}{e\alpha}\right]^{\gamma}} \cdot \sqrt{1-e^{-4\Pi\eta N_S}} \left(\lambda + \frac{0,5772}{\lambda}\right) \sqrt{\frac{\Pi \Gamma_{ss}(W_n)}{2\eta W_n^{3}}} \quad , \quad (1)$$

Where $E\{\}$ = expected value, η damping ratio and $T_n = 2\pi / W_n$: natural period.

This method considers the amplitude variation of ground acceleration with time, the strong motion duration and the frequency content.

In Eq. (1) β is the intensity parameter and α and γ are the shape parameters of the chi-square function for the expected square ground acceleration defined by Saragoni (Ref. 4).

$$E\{\ddot{u}_g^2(t)\} = \beta e^{-\alpha t} t^{\gamma},$$

Where $\ddot{u}_g(t)$ ground acceleration and t : time

In Eq. (1) N_S represents the strong motion duration of the earthquake Δt_s measured in number of natural periods T_n :

$$N_S = \frac{\Delta t_s}{T_n} \tag{3}$$

with

$$\Delta t_s = \frac{2\sqrt{\gamma}}{\alpha} \tag{4}$$

The value of λ of Eq.(1) is given by

$$\lambda = \sqrt{2 \mathrm{Ln}\,(N_{ES})} \tag{5}$$

with :

$$N_{ES} = N_S(1 + \frac{1}{4\pi\eta\, N_S} \mathrm{Ln}(0.18 + e^{-4\pi\eta N_S}) \tag{6}$$

The frequency content of the earthquake accelerogram is given in Eq. (1) by the spectral density function T_{ss} (w)

$$\Gamma_{ss}(w) = S_o\, e^{-QW}\, W^P$$

with

$$S_o = \frac{Q^{P+1}}{\Gamma(P+1)}$$

where

$$\Gamma(\gamma+1) = \int_o^{\infty} e^{-t}\, t^{\gamma}\, dt \quad \text{is the gamma function.}$$

The parameters P and Q can be estimated considering the intensity of zero crossings ν_O and the intensity of maxima ν_m of the accelerogram :

$$\nu_0 = \frac{1}{\pi Q} \sqrt{(P+1)\ (P+2)} \tag{9}$$

$$\nu_m = \frac{1}{2\pi Q} \sqrt{(P+3)\ (P+4)} \tag{10}$$

The parameters α and γ can be estimated in Chile in hard soil using the following relationship

$$\gamma = \left(\frac{10}{\Delta t_s} + 1\right)^2 \tag{11}$$

$$\alpha = \frac{2}{\Delta t_s} \left(1 + \frac{10}{\Delta t_s} \right)^2 \tag{12}$$

with

$$\Delta t_s = \begin{cases} 2 \times 10^{-4}\, e^{1.51\, M_S} - 2.1 \times 10^{-3}\, M_S\, (D-60) & ;\ D > 60 \text{ Km.} \\ 2 \times 10^{-4}\, e^{1.51\, M_S} & D < 60 \text{ Km.} \end{cases} \tag{13}$$

where M_S Richter magnitude and D = epicenter distance (Km).

The intensity parameter is estimated by

$$\beta = \frac{E\ \{W_A(t_o)\}^{\alpha^{\gamma}+1}}{\Gamma(\gamma+1)}$$

where $E\{W_A\ (t_o)\}$ is the expected energy of the earthquake which can be estimated using the following attenuation formula

$$E\ \{W_A(t_o)\} = 2.5 \times 10^{-3}\, e^{2.77\, M_s}\, D^{-0.25\, M_s} \tag{15}$$

for intermediate depth chilean earthquakes.

AVERAGE RESPONSE OF VIÑA DEL MAR

The S 20° W (NS) component of Viña del Mar is characterized by the following parameters:

$\alpha = 0{,}149 \ |\mathrm{sec}^{-1}$ $\quad \beta = 0.122 \times 10^{-6} \ |g^2 \mathrm{sec}^{-\gamma}| \quad \gamma = 4{,}63$

$\Delta t_s = 28{,}89 \ |\mathrm{sec}| \quad \nu_0 = 6{,}91 \ |\mathrm{crossing/sec}| \quad \nu_m = 7.49 \ |\mathrm{max/sec}|$

$P = 31{,}222 \times 10^{-2} \quad Q_G = 8.028 \quad 10^{-2} \quad S_0 = 4.078 \times 10^{-2}$

Substituting these values in Eq. (1) and considering $\eta=0.05$ the average displacement response spectra is estimed for Viña del Mar. The expected absolute acceleration response spectra $E\{S_A(T_n, \eta)\}$ can be estimated considering

$$E\{S_A(T_n,\eta)\} = \left[\frac{2\pi}{T_n}\right]^2 E\{S_D(T_n,\eta)\} \qquad (16)$$

Eq. (16) is only valid for $T_n \neq 0$. For $T_n = 0$.

$$E\{S_A(T_n, \beta)\} \rightarrow \quad E\{\ddot{u}_g(t)\}_{max}$$

In Fig. 2 the estimated average acceleration response spectrum for $\eta = 0{,}05$ of Viña del Mar S 20° W is compared with the measured spectrum. The agreement between both spectra is satisfactory. The method will be also used to estimate the acceleration response for $M_S = 8.5$ due to this satisfactory result.

EARTHQUAKE MECHANISM REDUCTION FACTOR

Comparing the results of the average response spectrum $\eta = 0.05$ obtained for the Viña del Mar for the 1985 earthquake, with the values of the Chilean Code NCh.433.Of.72 it is concluded that methods which considers reduction of elastic spectra by elastoplastic behavior such Newmark Hall method (Ref. 5) can not explain the seismic behavior of Viña del Mar buildings. These buildings almost respond in the elastic range (Ref. 2). Since it is not possible to explain the satisfactory behavior of these buildings considering important nonlinear incursion, it is defined a new factor of reduction R_T due to the characteristics of the source of the earthquake. This factor is in independent to the properties.

If R_μ is the reduction factor due to the provide ductility μ, then the total reduction factor R is

$$R = R_T \cdot R_\mu \tag{17}$$

The factor RT was estimated dividing the average acceleration response spectra of S 20° W Viña del Mar component by S_e the spectra of the Chilean Code for soil type. ($T_o = 0.30$ sec).

$$S_e = \begin{cases} 0.10\ g & ; \quad T < 0.30 \text{ sec.} \\ 0.10\ g \dfrac{2T\ T_o}{T^2+T_o^2} & ; \quad 0.30 \text{ sec} < T < 1.17 \text{ sec.} \\ 0.048\ g & ; \quad T > 1.17 \text{ sec.} \end{cases} \tag{18}$$

The values of RT obtained are shown in Fig. 3.

Using these values of RT the design code was estimated at Viña del Mar for an earthquake MS = 8.5 and $\mu = 4.5$ for shear wall reinforced concrete buildings.

The spectrum found is given by

$$S_A(T_n) = \begin{cases} 0.14\ g & ; \quad 0 < Tn < 0.20 \\ \dfrac{1.12\ g}{40\ Tn} & ; \quad 0.20 < Tn \end{cases} \tag{19}$$

The spectrum and the chilean code one for ultimate design conditions are compared in Fig. 4. The chilean code shows to be on the safe side for epicentral condition, An increment of 20% may be necessary in the range of Tn < 0.20 sec. corresponding to building less than 6 stories. From this results a reduction factor RT = 5 is recommended.

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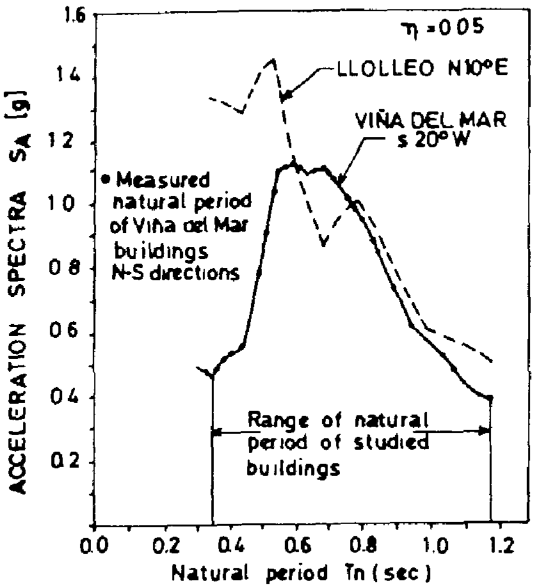

FIG 1 COMPARISON BETWEEN ABSOLUTE ACCELERATION RESPONSE SPECTRA OF LLOLLEO AND VIÑA DEL MAR RECORDS EARTHQUAKE 1985

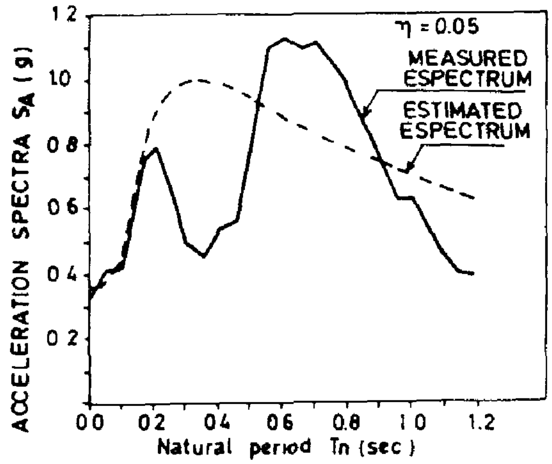

FIG. 2 COMPARISON BETWEEN MEASURED AND ESTIMATED ACCELERATION RESPONSE SPECTRA VIÑA DEL MAR s20°W, MARCH 3, 1985

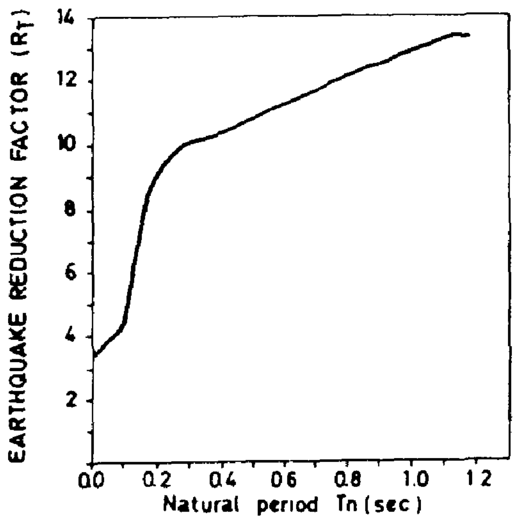

FIG 3 REDUCTION FACTOR R_T DUE TO EARTHQUAKE MECHANISM

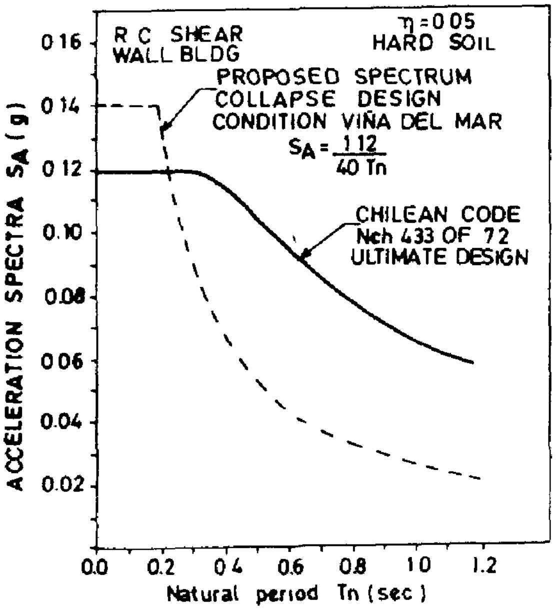

FIG 4 COMPARISON BETWEEN CHILEAN CODE AND PROPOSED COLLAPSE CODE FOR VIÑA DEL MAR HARD SOIL