First European Conference on Earthquake Engineering and Seismology
(a joint event of the 13th ECEE & 30th General Assembly of the ESC)
Geneva, Switzerland, 3-8 September 2006
Paper Number:165

DAMAGE EVALUATION IN R/C SHEAR WALLS USING THE DAMAGE INDEX OF PARK & ANG

Claudio OYARZO VERA[1]

SUMMARY

The damage indexes are a concept introduced in the last years by some researchers, these had the particularity that they can measure the damage on a defined scale (0 to 1). These indexes have become an important tool for the evaluation of damage on structures, but also they could became a design variable in performance based design.

One of the most used is the Damage Index of Park & Ang, which considers the damage due to deformations over the elastic stage and the cumulative effect of reversible loads.

This index was made to quantify the damage in slender structural elements (beams or columns) which generally fail by flexion, this situation establishes a limitation for its use in buildings constructed according to the Chilean practice, because they are generally structured with shear walls, which present a low slenderness.

This research intends to show that it is not possible to use the equations proposed by Park & Ang directly on non-slender elements, because the parameter β included in this Damage Index depends on other variables, which are not considered by the authors. It is possible to detect important differences in the values of parameter β and in the damage index, itself.

Some of these variables would be: the loads history, the degradation level, the type of fault observed in the testing wall and the cumulative ductility of displacement recorded during the test.

Another interesting fact is that the values of β obtained using experimental data, seems to be more related to the cumulative ductility recorded on the failure, than the displacement reached at the same situation.

All this facts show that it is necessary to obtain a new equation for β to represent in a better way the behavior of non-slender elements.

1. INTRODUCTION

The high seismic loads applied over structures in several situation force them to get in the non-elastic state, where the behavior of the structural elements depends on the energy dissipation capacity and high demands of ductility resistance. These irruptions in a level of deformation superior to capacities of the structures are associated to a certain level of damage in its elements.

The structural damage is an idea which is always present in the mind of the engineers. Yet, there is still no agreement in its conceptualization and how to turn it into a quantifiable variable, in spite of its importance.

[1] Universidad Católica de la Santísima Concepción. Alonso de Ribera 2850. Casilla 297. Concepción. CHILE
Email: coyarzov@ucsc.cl

The Chilean codes in use at present define structural damage as " a loss or deterioration of some structural characteristics, such as resistance, rigidity, ductility, mass or system of foundations". On another paragraph states that, "the damage caused by an earthquake can be classified in three categories: light, moderate and severe, based on a qualitative analysis of the structure according to established criteria and under responsibility of a specialist professional" [INN, 1996].

This lack of precision and the requirements imposed by the implementation of Performance-Based Design demand researchers the creation of mathematical tools which help to define the damage in an objective way and through quantifiable physical parameters.

There are a series of measurable response parameters associated to the damage level observed on structural element, for example deformation, ductility, drift, acceleration, plastic energy dissipation, hysteresis energy dissipation, etc.

One of the fundamental ideas of performance-based design postulates that limiting these parameters it would be possible to control the damage produced on the structure. So, it would be necessary to establish limits for maximum or cumulative demands of several parameters. Also, to supply mechanical characteristics to the structure which help to control its response within the established limits [Teran-Gilmore, 1997]. From this, the concepts of damage evaluation is born.

The damage index (*DI*) is a concept introduced the past years by some researchers as a specific damage measurement tool. They are mathematical functions whose variables are one or several of the structural parameters mentioned before, and they quantify the structural damage in a determinated scale between the value zero (situation without damage) and one (collapse). In the case of reinforced concrete, non-recoverable deformations (over the elastic state), displacement ductility and energy dissipated by hysteresis are used as damage parameter, because they are easy to evaluate.

In the future, damage indexes could become important tools for damage evaluation, for making decision regarding to repairs, reinforcement or demolition of structures as well as a design variable under the criteria of structural performance.

2. PROBLEM DEFINITION AND OBJECTIVES

At present, there are a series of damage indexes, they have been proposed by different authors, who have obtained them from different experimental studies. One of the most used is the damage index of Park & Ang [Park & Ang, 1985]. The variables used by this index to quantify the damage are the displacements superior to the elastic rank and the energy dissipated by the hysteresis cycles. This index was elaborated in order to assess the damage in slender elements (beams or columns) which generally fail by flexion.

This condition establishes limitations for its use in buildings constructed according to the Chilean practice. It is well known, that most of the Chilean constructions are structured based of shear walls, which usually present low slenderness and, as its name indicates, its failure is dominated by shear. This paper tries to demonstrate that the direct application of the equations proposed by Park & Ang to shear walls, is not advisable and that they require corrections to be used with non-slender elements.

3. THEORETICAL FUNDAMENTS

The theoretical base which sustains this research mainly corresponds to the expressions proposed in 1985 by Alfredo Ang and Young-Ji Park [Park & Ang, 1985] to define the damage index that takes their names. This Damage Index has became one of the most widely used tools in damage evaluation, because of its simplicity, stability and large experimental endorsement.

This index is constructed based on a linear combination between the maximal normalized deformation and the energy dissipated in the cycles of hysteresis. This combination is presented in equation (1):

$$DI = \frac{u_{Max}}{u_{Mon}} + \beta \cdot \frac{E_H}{F_y \cdot u_{Mon}} \quad (1)$$

where:

u_{Max}	:	Maximal deformation reached due to reversible load test.
u_{Mon}	:	Ultimate deformation reached due to monotonic load test.
F_Y	:	Yield strength
E_H	:	Energy dissipated in the cycles of hysteresis
β	:	Non-negative Parameter.

Based on experimental data, they determinated that, for slender elements, the parameter β it is function of confinement ratio (ρ_w), shear span ratio (ℓ/d),longitudinal reinforcement ratio (ρ_t) and normalized axial force (n_0). The expression proposed by Park & Ang to β is in equation (2):

$$\beta = 0.7^{\rho_w}\left(-0.447 + 0.073 \cdot \ell/d + 0.24 n_0 + 0.314 \rho_t\right) \tag{2}$$

Note that the parameter β does not depend on load history.

Based empirical studies, the authors affirm that the equation (2) behaves well under the following conditions:

$$\begin{aligned} 0.2 < \rho_w < 2.0 \\ 1.0 < \ell/d < 6.6 \\ 0 \le n_0 < 0.52 \\ 0.04 < \rho_t < 0.45 \\ 160\ kg/cm^2 \le f'_c < 415\ kg/cm^2 \end{aligned} \tag{3}$$

The experimental results also indicates that the parameter β varies between -0.3 and 1.2, generally adopting a value near 0.15. However, it is necessary to underline that the damage index cannot adopt a negative value, because by definition its value must be between 0 and 1.

If $\beta=0$, the collapse of the element is produced by excessive deformation. If $\beta=0.6\sim0.8$, the collapse of the element is produced by energy dissipation (cumulated damage).

Table 1 shows the value adopted by the Damage Index of Park & Ang due to different level of damage [Williams & Sexsmith, 1995].

Table 1: Park & Ang Damage Index Performance

$ID_{P\&A}$	Level Damage	
0.00 - 0.10	No damage	- Localized minor cracking at worst
0.10 - 0.25	Light damage	- Minor cracking throughout.
0.25 - 0.40	Moderate damage	- Severe cracking and localized spalling.
0.40 - 1.00	Severe damage	- Crushing of concrete and exposure of reinforcing bars.
1.00 and more	Collapse	- Total failure of the structure.

4. EXPERIMENTAL BASIS

The experimental basis which endorse this research corresponds to the data collected from 12 reinforced concrete walls tested in 1999 in the Universidad Técnica Federico Santa Maria in the context of a project funded by the “Fondo Nacional de Desarrollo Científico y Tecnológico” of the Chilean Goverment [Leiva, Bonelli et Al., 1999].

These walls were designed in order to obtain an initial resistance to shear high enough to develop yield in flexion, but simultaneously small enough so that the degradation allowed a failure by shear previous to the failure by flexion.

The walls were designed with a rectangular section, without border elements. The thickness of the walls was 10 (cm), the length was 80 (cm) and the height was 150 (cm). They were mounted on a rigid beam to provide embeding conditions and another rectangular (20x20 cm) beam was constructed on the top, whose function was to transmit the lateral load to the wall (Figure 1).

The longitudinal reinforcement (flexion) was the same for all the walls. It was formed by three 12(mm) diameter bars triangularly arranged in each end of the section and a row of 8 (mm) diameter bars in the web of the wall. In addition, triangular stirrups were used to link the three 12 (mm) bars of the ends. In some cases, 6 (mm) diameter rectangular stirrups were used for linking the extreme bars with the first bar of the web in order to provide confinement to longitudinal reinforcement at ends.

Finally, the transversal reinforcement was different in each wall, to obtain different resistance to shear. For this, 5.5 (mm) diameter bars anchored to edge reinforcement with 180º hooks were used. Its spacing was variable to obtain different ratios of cross-sectional reinforcement.

Each wall was tested in a reaction frame anchored to the floor. The load pattern was applied to them on the top-beam by a double action load cell with 20 (T) of capacity.

Each sample was wired to measure displacements, angular deformations and loads. These data were automatically stored on a computer.

Walls M5 and M6A were loaded monotonically until the failure. All the other walls were tested under reversible load.

The load pattern used began with a series of three cycles of equal magnitude, then they progressingly increased until reaching the cracking by flexion. After this point, each load history series began with a first stage (degradation cycles) whose function is to degrade the resistance and stiffness of the element. This degradation cycles group was composed by four cycles which decrease one rate of 25%. The second stage (stabilization cycles) tried to stabilize the response of the walls, they included three cycles of equal magnitude to the first of the previous degradation cycle.

This series was repeated continuously, increasing the cycles amplitude and following the pattern already described until reaching the failure. Walls M1, M2B, M6B and M7B were tested with a similar load pattern shown in Figure 2. Wall M3B was tested with a load pattern where the stabilization cycles were omitted. Walls M2A, M3A, M4, M6C and M7A were tested with a load pattern with approximately the double or triple of cycles than the standard test in order to obtain a greater shear resistance degradation.

A summary of the most important characteristics of reinforcement appears in Figure 1 and Table 2. The Standard load pattern appears in Figure 2.

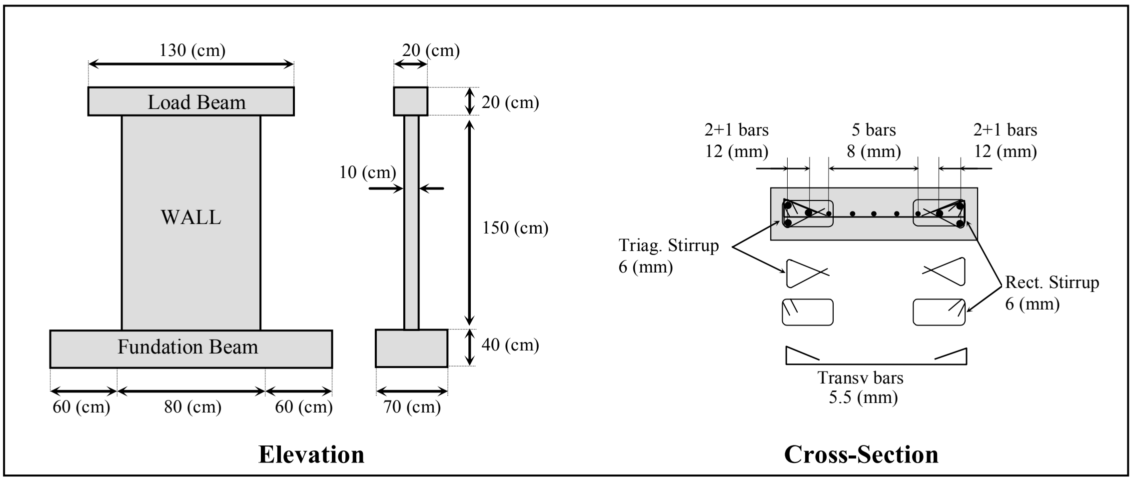

Figure 1: Walls' Geometry and Reinforcement Details.

Table 2: Walls' Reinforcement and Load Test Patterns

Wall	Edge Longitudinal Reinforcement	Web Longitudinal Reinforcement	Web Transversal Reinforcement	Transv. Reinf. Ratio %	Load Pattern
M1	3 bars of 12 (mm)	5 bars of 8 (mm)	Bars of 5.5 (mm) @ 8 (cm)	0.300	Standard
M2A	3 bars of 12 (mm)	5 bars of 8 (mm)	Bars of 5.5 (mm) @ 12 (cm)	0.200	Double Standard
M2B	3 bars of 12 (mm)	5 bars of 8 (mm)	Bars of 5.5 (mm) @ 12 (cm)	0.200	Standard
M3A	3 bars of 12 (mm)	5 bars of 8 (mm)	Bars of 5.5 (mm) @ 14 (cm)	0.171	Double Standard
M3B	3 bars of 12 (mm)	5 bars of 8 (mm)	Bars of 5.5 (mm) @ 14 (cm)	0.171	Only degradation Cycles
M4	3 bars of 12 (mm)	5 bars of 8 (mm)	Bars of 5.5 (mm) @ 25 (cm)	0.096	Standard with triple degradation
M5	3 bars of 12 (mm)	5 bars of 8 (mm)	No Reinforced	0.000	Monotonic
M6A	3 bars of 12 (mm)	5 bars of 8 (mm)	Bars of 5.5 (mm) @ 25 (cm)	0.096	Monotonic
M6B	3 bars of 12 (mm)	5 bars of 8 (mm)	Bars of 5.5 (mm) @ 25 (cm)	0.096	Standard
M6C	3 bars of 12 (mm)	5 bars of 8 (mm)	Bars of 5.5 (mm) @ 25 (cm)	0.096	Standard with triple degradation
M7A	3 bars of 12 (mm)	5 bars of 8 (mm)	Bars of 5.5 (mm) @ 20 (cm)	0.119	Double Standard
M7B	3 bars of 12 (mm)	5 bars of 8 (mm)	Bars of 5.5 (mm) @ 20 (cm)	0.119	Standard

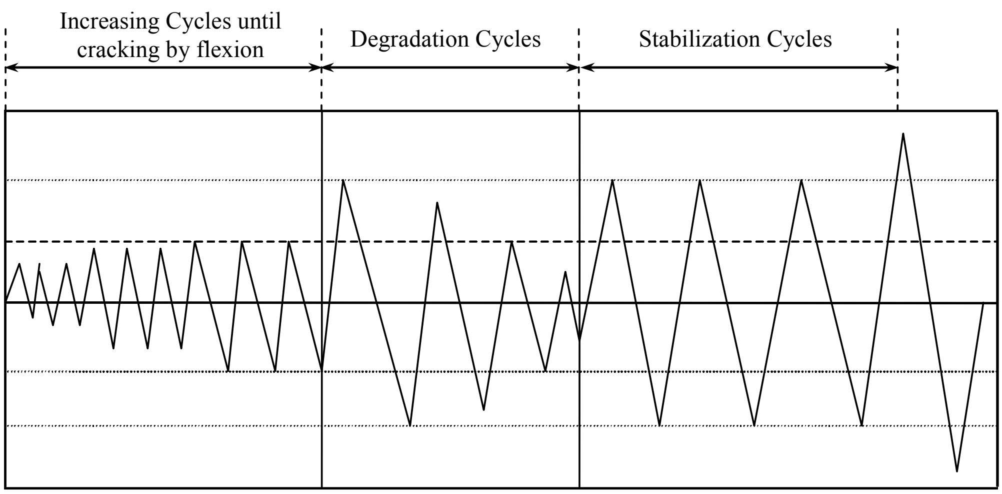

Figure 2: Standard Load Pattern

For every wall, a complete record of the response, in terms of the load-displacement cycles on the top of the wall was obtained. As an example, the records for wall M1 and M3A were in figure 3 and figure 4., both were tested under reversible load.

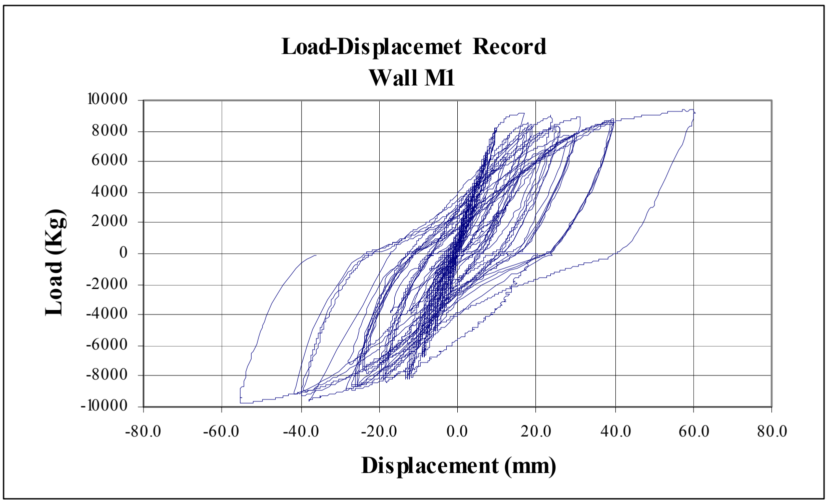

Figure 3: Load-Displacement Record Wall M1

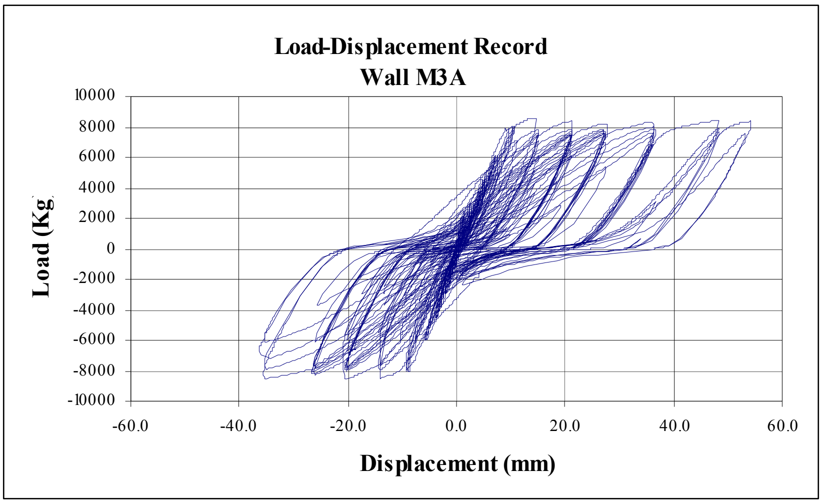

Figure 4: Load-Displacement Record Wall M3A

The types of failure of each wall are indicated in table 3. In this table, the parameter u_{max} represents the maximal displacement registered at the top of the wall and μ_c corresponds to the cumulated displacement ductility of the wall. In other words, the sum of all maximal displacements registered at the top after the yield of the longitudinal reinforcement normalized by the yield displacement measured in each wall.

Table 3: Experimental Results and Observed Failure.

Wall	Failure Cycle	u_{max} [mm]	μ_c	Observed Failure
M1	59	50.4	258.0	Flexion.
M2A	81	25.8	203.0	Shear - Diagonal Compression.
M2B	59	29.2	165.5	Shear - Diagonal Compression.
M3A	70	39.0	225.3	Shear - Diagonal Compression.
M3B	52	37.4	89.5	Shear - Diagonal Tension.
M4	65	33.7	351.8	Shear – Web Destruction
M5	--	49.0	4.8	Shear.(Monotonic Test)
M6A	--	91.7	19.1	Flexion (Monotonic Test)
M6B	50	32.4	125.8	Shear - Diagonal Compression.
M6C	--	--	--	Sliding.
M7A	73	31.8	285.8	Shear - Diagonal Compression.
M7B	57	40.0	148.5	Shear – Diagonal Compression and Tension.

Walls M5 and M6A, monotonically loaded, and Wall M6C, which showed an abnormal type of failure (base sliding), will not be considered in the next analyses, because this phenomena are not included in this research. Only Wall M6A is considered as the monotonic reference test in Park & And Damage Index calculation.

5. RESULTS ANALISYS

5.1 Evaluation of β According to Park & Ang Expression

As the geometric and mechanical characteristics of tested walls are known, it is possible to calculate parameter β using the equation proposed by Park & Ang (Eq. 2). It is worth to notice that in general the values adopted by the variables of β are equal for all the walls. The only variable which changes is the transverse reinforcement ratio, that is assimilable to confinement ratio (ρ_w), because its function refering to the shear is similar to confinement reinforcement (stirrups) in slender elements. Therefore, it is possible to replace the confinement ratio with the transverse reinforcement ratio in the equation (2)

So, the parameters of equation (2) adopt the following values:

ℓ = 150 (cm)
d = 69.5 (cm)
n_0 = 0.2 (model restriction)
ρ_t = 1.16%
ρ_w = *0.018~0.300%*

The values of the parameter β calculated according to expressions of Park & Ang (β_T) for each of the tested walls appear in Table 4. In addition, there are indicated the values of transverse reinforcement ratio (ρ_w) and the values of Park & Ang damage index (DI_T) calculated using Eq (1) and β_T.

The results show that transverse reinforcement ratio (ρ_w) does not have great incidence in the value of β_T . In fact, however ρ_w displays variations of nearly 100% in some cases, the value of β_T does not show greater differences.

5.2 Experimental Determination of β

Since the record and binnacle of every tests are available, it is possible to know exactly the failure point of the elements, with its respective displacement and hysteresis energy records associated.

If these values are used in the equation (1) and the condition of collapse is imposed, that is, the damage index must be equal to 1.0 at the failure point, it is possible to obtain the experimental value of β (β_E), these values are also shown In table 4.

Table 4: Results Summary

Wall	ρ_w %	u_{max} [mm]	μ_c	β_T	β_E	DI_T	DI_E	Observed Failure
M1	0.300	50.4	258.0	0.1093	0.060	1.380	1.00	Flexion
M2A	0.200	25.8	203.0	0.1132	0.115	0.994	1.00	Shear - Diagonal Compression.
M2B	0.200	29.2	165.5	0.1132	0.145	0.857	1.00	Shear - Diagonal Compression.
M3A	0.018	39.0	225.3	0.1209	0.125	0.978	1.00	Shear - Diagonal Compression.
M3B	0.018	37.4	89.5	0.1209	0.130	0.970	1.00	Shear - Diagonal Tension.
M4	0.096	33.7	351.8	0.1175	0.095	1.150	1.00	Shear – Web Destruction
M6B	0.096	32.4	125.8	0.1175	0.145	0.880	1.00	Shear - Diagonal Compression.
M7A	0.119	31.8	285.8	0.1166	0.120	0.977	1.00	Shear - Diagonal Compression.
M7B	0.119	40.0	148.5	0.1166	0.095	1.120	1.00	Shear - Diagonal Comp. and Tension.

5.3 Results Comparison

Figure 5 presents the theoretical and experimental values of β for each wall. The differences between the values of the parameter β calculated with the equation (2) used in slender elements (β_T) and the parameter β calculated by the experimental data (β_E) are clearly shown on this graphic.

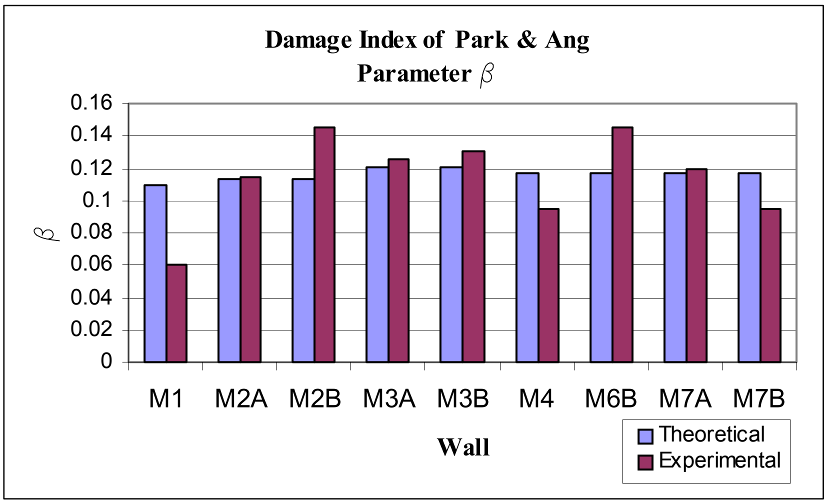

Figure 5: Parameter β of Park & Ang

Also, it has been demonstrated that it is not possible to assign an single value of β to all walls. In fact, the experimental value of β oscillates between 0.060 and 0.145, showing manifest differences between those walls whose failure was dominated by flexion and those that failed by shear (Table 4)

In addition, it is possible to be noticed that value of β_E is more related to the cumulative displacement ductility at collapse point, than maximal displacements measured at same point. High levels of cumulative displacement ductility are related to low values of β, because the effect of cycles dissipated energy are less important.

On other hand, if the walls are grouped based on the value of β_E, four groups of walls are possible to identify, as Table 5 shows.

Table 5: Walls grouped according to β_E

	β_E	Walls
Group I	0.060	M1
Group II	0.095	M4-M7B
Group III	0.120	M2A-M3A-M7A
Group IV	0.140	M2B-M3B-M6B

There is in Group I just one wall (M1) that displays the smaller value of $\beta_E = 0.060$. In addition it shows the largest difference between β_T and β_E. The failure mechanism of this wall was dominated by flexion, different to all other walls. It would have been expected that these wall present some kind of similarity between β_T and β_E, because the type of failure is similar to slender elements, which were used to calibrate the equation (2).

In Group II are included walls M4 and M7B, those correspond to a wall tested whit a standard with triple degradation cycles load pattern and a wall that presented anomalies in its stiffness, respectively.

In Group III are those walls submissive to double standard load pattern (M2A, M3A, M7A). Its value of β_E is quite closed to the value calculated by equation (2).

In Group IV are included the walls tested under standard load patterns (M2B, M6B) and tested just under degradation cycles (M3B). These walls presented the minors levels of cumulative displacement ductility.

From all the previous facts, it is possible to deduce that the equation proposed by Park & Ang to calculate the parameter β it is not valid for non-slender elements such as walls. In fact, it can be inferred that the value adopted by β not only depends on the structural characteristics of the element, but also of other variables, like for example, the load pattern applied to the walls, the type of failure and the level of cumulative displacement ductility.

This is very clear when the records of walls M1 and M3A are compared. Both reach similar displacement and ductility levels, but the experimental value of β (β_E) is very different. The explanation could be found by looking to their load-displacement graphics.

On the one hand, Wall M1 (Fig 3) presents hysteresis cycles quite stable and wide, typical of flexural dominated performance. This kind of cycles indicate a good capacity of energy dissipation, therefore the damage should be dominated by displacements. This agrees with the low value of $\beta_E = 0.060$.

On the other hand, Wall M3A (Fig 4) shows narrower cycles than Wall M1. This kind of cycles are characteristical of shear dominated performance. This situation is associated to low capacity of energy dissipation, therefore the damage will be more associated to the cumulative effect of reversible loads. In this case, the value of β_E should be greater, indicating higher relevance of the energy factor in Park & Ang damage index (Eq. 1).

6. CONCLUSIONS

Concerning the Index of Damage of Park & Ang, this research concludes that it is not feasible to assign an unique value of β to all walls, because there are notorious differences between walls whose failure is dominated by flexion and those whose failure is dominated by shear. This factor was not considered in the original formulation of the index.

In addition, it is evident that the equation propose by Park & Ang to calculate the parameter β is not applicable in the case of non-slender elements as walls. In fact, it can be infer that the value adopted by β depends on other variables not considered in the original formulation, like load pattern, type of failure and level of cumulative displacement ductility.

It is worth it to remark that the values of β obtained using experimental data seem more related to the cumulative displacement ductility recorded at failure condition than to displacements recorded at the same point.

All the previous assessment arrive to the conclusion that it is necessary to recalibrate the parameter β and to obtain a new equation which represents in a better way the behavior of this parameter in non-slender elements.

7. ACKNOWLEGMENT

This paper has been written based on the thesis whereupon the author obtained his degree of Civil Engineer in the Universidad Técnica Federico Santa María [Oyarzo, 2003] and summarizes only part of the researched topics. The support of this institution in the development of this research is thanked. Specially, the author wants to thank the help and guidance of Professor Gilberto Leiva H.

Finally, it is important to acknowledge the assistance of the Universidad Católica de la Santísima Concepción in the elaboration of this paper.

8. REFERENCES

Instituto Nacional de Normalización – Chile (1996), Diseño Sísmico de Edificios, *Norma Chilena Oficial 433 Of.96, INN, Chile.*

Leiva, G.; Bonelli, P.; Méndez, J.; Sanz, C.; Delgado, M. (1999), Resistencia al Corte de Muros de Hormigón Armado sometido a Altas Demandas de Ductilidad. *Informe Final Proyecto Fondecyt Nº 195039, UTFSM*, *Valparaíso - Chile.*

Oyarzo, C. (2003), Evaluación Del Daño En Muros De Hormigón Armado Sometidos A Altas Demandas De Ductilidad. *Memoria para optar al título de Ingeniero Civil, UTFSM*, *Valparaíso - Chile*.
Park, Y-J.; Ang, A. (1985) Mechanistic Seismic Damage Model For Reinforced Concrete, *Journal of Structural Engineering, 111, n° 4, 722-739.*
Teran-Gilmore, A. (1997), Energy Concept and Damage Indices, *Proceeding of Symposium in Honor of Vitelmo Bertero, Berkeley, California Jan. 31-Feb. 1, 1997. http://nisse.berkeley.edu/lessons/teran-gilmore.html*
Williams, M.; Sexsmith, R. (1995), Seismic Damage Indices for Concrete Structures: A State-of-the-Art Review, *Earthquake Spectra, 11,n° 2,319-249.*