A seismic vulnerability index for confined masonry shear wall buildings and a relationship with the damage

Kenneth A. Gent Franch [a], Gian M. Giuliano Morbelli [b], Maximiliano A. Astroza Inostroza [c], Roberto E. Gori [d,*]

[a] *University of Concepcion, Chile*
[b] *Department of Civil Engineering. University of Concepcion, Chile*
[c] *Department of Civil Engineering. University of Chile, Chile*
[d] *Department of Construction and Transportation Engineering, University of Padova, Italy*

Abstract

In this paper the Italian CNR-GNDT vulnerability index for masonry buildings was modified to apply in confined masonry buildings and to obtain a reasonable relationship with the wall density per unit floor index. With this purpose, a sample of twenty-four confined masonry buildings with three and four storeys built during the last twenty-five years for social housing programs was used. A relationship has also been obtained between the value of the proposal index and the damage observed in the March 1985 Central Chile subduction earthquake ($M_s = 7.8$).

Keywords: Seismic vulnerability; Confined masonry buildings; Relationship vulnerability index–damage

1. Introduction

Given that seismic prediction is still far from becoming a reality, it is necessary to improve the prognosis of the seismic behaviour of existing structures. This is the reason why studies of *Seismic Vulnerability of Buildings* have been developed to evaluate the expected damage in the different types of buildings when there is a severe earthquake – magnitude greater than 7.0 in the subduction zone of Chile – in their area.

With this purpose, several methods have been proposed in order to qualify the seismic behaviour of many structures built in US. An example is the fast visual inspection method (RVS) to identify the buildings with higher seismic risk [14]. Also, in Latin America some researches have been developed to evaluate the seismic performance of buildings, making use of the international experience such as the VISION 2000 Committee publications, the NEHRP guides, the ATC-40, the program HAZUS 99, FEMA 273 and 274. As an example, Aguiar [2] developed a software (CEINCI3) to simulate the structural seismic behaviour and to predict the damage in 36 types of buildings that may be located in any city of Venezuela, Colombia, Ecuador or Peru.

Experience accumulated during a destructive earthquake shows that the seismic behaviour within one type of building is not uniform in an area with the same soil conditions. This emphasizes the importance of having a tool that can quantify the seismic behaviour of a specific type of building in order to anticipate the damage level during a destructive earthquake.

To contribute towards this, a seismic vulnerability index for confined masonry shear wall buildings with three and four storeys has been obtained by modifying the vulnerability index of the method developed by the Italian CNR-GNDT for stone unreinforced masonry structures in southern Europe [11,12].

The Italian method has been chosen considering that the confined masonry structures were built for the first time in the reconstruction of the buildings destroyed by the 1908 Messina Italian earthquake [13]. In addition, the experience of Italian researchers applying the CNR-GNDT method to masonry buildings is ample [22,8–10,5]. The method is very

* Corresponding author. Tel.: +39 0498275595; fax: +39 0498275604.
E-mail addresses: kennethgent@gmail.com (K.A. Gent Franch), ggiulian@udec.cl (G.M. Giuliano Morbelli), mastroza@ing.uchile.cl (M.A. Astroza Inostroza), roberto.gori@unipd.it (R.E. Gori).

easy to apply and it considers the structural characteristics of the buildings in more detail than other indexes, for example *the wall density* index. This latter index, proposed by Meli [21] and Astroza et al. [6], was related to the seismic damage of confined masonry shear wall buildings during a destructive earthquake.

The US methods have not been used because the confined masonry buildings are not built in this country. As an evidence of this, the performance report of masonry structures in the 1994 Northridge earthquake does not include any commentaries about these buildings [27] and the confined masonry structure does not consider between the building types which RVS method [14] is applied.

In Chile, the Italian methodology was first applied by Sáez [26]; he used it to evaluate the seismic vulnerability of adobe houses built in southern Chile. Several studies have since been developed in order to apply the CNR-GNDT method to more popular structural types, such as reinforced concrete and confined masonry shear wall buildings [4,1,3,20,15].

2. Seismic behaviour and characteristics of confined masonry shear wall buildings

The use of confined masonry shear wall buildings started during the 1940s, due to the good behaviour observed in dwellings built with this type of reinforcement during the 1939 Chillán earthquake, $Ms = 7.8$ [23]. Prior to this, masonry buildings had been built with very thick walls using handmade clay bricks and mortar lime, without reinforcement and in the style of European neoclassical architecture. These non-reinforced masonry buildings were badly damaged during the 1906 Valparaiso and 1928 Talca earthquakes.

Confined masonry buildings are regular with regard to both plan and elevation and are configured mainly with shear walls tied together at floor levels by reinforced concrete beams that form part of the confinement element. At least two lines of walls are present along each principal plan direction; along the longitudinal direction they are located at the perimeter whereas there is also a median wall along the transverse direction. Wall thickness varies between 140 and 200 mm depending on the size of the masonry units. Floors generally consist of cast-in-place reinforced concrete slabs between 100 and 120 mm thick. Storey height varies between 2300 and 2400 mm.

Confined masonry buildings consist of load-bearing unreinforced masonry walls commonly made of clay bricks or concrete blocks, confined by cast-in-place reinforced concrete vertical tie columns as shown in Fig. 2.1. These tie columns are located at regular intervals, no greater than 6000 mm, and are connected together with reinforced concrete horizontal tie beams, located at floor level; both R.C. elements are cast after the masonry walls are built. Tie columns and tie beams prevent damage due to out-of-plane bending effects and improve wall ductility. Tie columns have a rectangular section with dimensions typically corresponding to the wall thickness (150–200 mm) and depth of 200 mm. Both tie columns and tie beams must have at least four 10 mm diameter longitudinal reinforcements; 6 mm diameter stirrups must be spaced 100 mm apart at the extremes (critical zone, see Fig. 2.1) and 200 mm at the

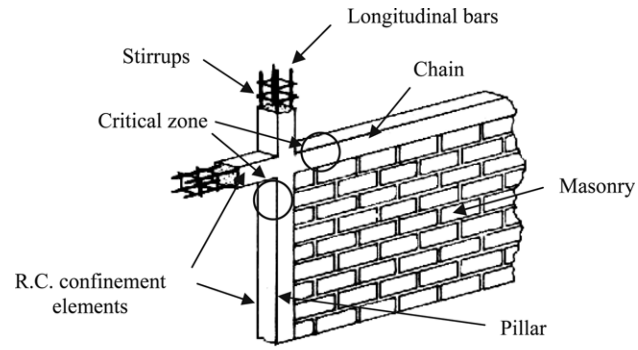

Fig. 2.1. Details of reinforcement of confined masonry shear wall.

centre of the elements. The stress method used for the design is according to NCh2123.Of97 Chilean code [17]. The typical masonry shear strength is 0.5–1.0 MPa.

A partially confined wall is sometimes used in one or two storey houses. In these cases, the wall is confined with a reinforced concrete column at one end and a tensile bar at the other end; this is frequently used when there are openings. In other cases the R.C. column is placed at the centre of the masonry wall, loosing the confinement effect. This situation has no major effect on the seismic behaviour when wall density is high – 3% or more in each plan direction – as verified near the epicentre of the 1985 Central Chile earthquake. Nevertheless, in buildings with three or four storeys, the lack of tie columns at one or two ends of the wall is responsible for most of the more serious damage to confined masonry walls, as Fig. 2.2 shows for a building located in the epicentral area of the 1985 Central Chile earthquake ($Ms = 7.8$). In addition, when the wall density per unit floor is low – 1% or less – the shear cracks propagate through the ends of the tie columns as shown in Figs. 2.2 and 2.3. This corresponds to the critical zone, see Fig. 2.1.

3. Seismic vulnerability of confined masonry buildings

In order to quantify the consequences of an earthquake, it is necessary to know the vulnerability or physical deterioration (damage) that the building suffers during the event. The vulnerability is a characteristic of the building and varies according to building type. Different indexes or methodologies have been proposed to qualify the seismic structural vulnerability, but few have been related with confined masonry buildings and the damage to these buildings after a severe earthquake. These relations are clearly of an empirical nature since they must be obtained from surveying the affected area after a destructive earthquake.

In the last fifteen years, wall density, d, calculated as wall area in each direction, divided by floor area, has been the most often used index to characterize confined masonry buildings [21,6,19], correlating the wall density per unit floor, d/n, with the damage masonry buildings suffered during an earthquake. Astroza et al. [6] correlated the same index with the level of damage, defined in Table 3.1, which occurred in the March 1985 Central Chile earthquake in a sample of three and four storey buildings, complementing this sample with

(a) Global view of the building. (b) Crack pattern in the masonry panel.

Fig. 2.2. Damage to confined masonry building in Llolleo, Central Chile earthquake of March 3rd 1985.

(a) Building in Melipilla city. (b) Building in Santiago city.

Fig. 2.3. Damage to confined masonry walls, Central Chile earthquake of March 3rd 1985.

Table 3.1
Damage categories [6]

Category	Damage extension	Action to take
0. No damage	No damage in non-structural or structural members.	It is not necessary to take any action.
1. Light non-structural damage	Fine cracks on plaster, falling of plaster in limited zones.	It is not necessary to evacuate the building. Only architectural repairs are needed.
2. Light structural damage	Small cracks on masonry walls, falling of plaster block in extended zones. Damage to non-structural members, such as chimneys, tanks, pediment, cornice. The structure resistance capacity has not been reduced noticeably. Generalized failures in non-structural elements.	It is not necessary to evacuate the building. Only architectural repairs are needed in order to ensure conservation.
3. Moderate structural damage	Large and deep cracks in masonry walls, widely spread cracking in reinforced concrete walls, columns and buttress. Inclination or falling of chimneys, tanks, stair platforms. The structure resistance capacity is partially reduced.	The building must be evacuated and raised. It can be reoccupied after retrofitting. Before architectural treatment is undertaken structural restoration is needed.
4. Heavy structural damage	Wall pieces fall down, interior and exterior walls break and lean out of plumb. Failure in elements that connect buildings portions. Approximately 40% of essential structural elements fail. The building is in a dangerous condition.	The building must be evacuated and raised. It must be demolished or major retrofitting work is needed before being reoccupied.
5. Collapse	Partial or total collapse of building.	The building must be demolished and rebuilt.

the data collected by Meli [21]. Table 4.7 shows the results for the Chilean case; in order to avoid damage d/n must be greater than 1.15% when the seismic intensity is degree VIII in the Mercalli Modified Scale. This simple method allows preliminary evaluation of the seismic vulnerability of new or existing confined masonry buildings with the properties found on the west coast of Latin America.

4. Vulnerability method of CNR-GNDT

A more elaborate procedure to qualify the seismic vulnerability of buildings has been established by the Gruppo Nazionale per la Difesa dai Terremoti of the Italian Consiglio Nazionale delle Ricerche (CNR-GNDT). The method evaluates the seismic vulnerability of buildings, determining a *normalized index of vulnerability*, which is obtained with the aid of survey forms which are filled in for one building or a group [11,12]. The CNR-GNDT method was originally applied to stone rubblework buildings and reinforced concrete buildings, and a very detailed procedure exists for each one [7].

The method is based on the building characteristics, such as: type of construction, building's use, quality of materials,

Table 4.1
Vulnerability factors, class score and weight factor for stone masonry buildings [26]

Vulnerability factors	Class score A	B	C	D	Weight
1. Type and organization of earthquake resistant system	0	5	20	45	1
2. Earthquake resistant system's quality	0	5	25	45	0.25
3. Conventional strength capacity	0	5	25	45	1.50
4. Building location and foundations	0	5	25	45	0.75
5. Horizontal floor diaphragms' presence	0	5	15	45	1
6. Building plant configuration	0	5	25	45	0.50
7. Building elevation configuration	0	5	25	45	1
8. Maximum distance between walls	0	5	25	45	0.25
9. Type of roof	0	15	25	45	1
10. Non-structural elements	0	5	25	45	0.25
11. State of preservation	0	5	25	45	1

Table 4.2
Vulnerability factors, class score and weight factor for reinforced concrete buildings [26]

Vulnerability factors	Class score A	B	C	Weight
1. Organization of earthquake resistant system	0	6	12	1.0
2. Earthquake resistant system's quality	0	6	12	0.5
3. Conventional strength capacity	0	11	22	1.0
4. Building location and foundations	0	2	4	0.5
5. Horizontal floor diaphragms' presence	0	3	6	1.0
6. Building plant configuration	0	3	6	0.5
7. Building elevation configuration	0	3	6	1.0
8. Critical elements connection	0	3	6	0.75
9. Type of roof	0	3	6	1.0
10. Non-structural elements	0	4	10	0.25
11. State of preservation	0	10	20	1.0

Table 4.3
Modification of vulnerability factors, class score and weight factors for reinforced concrete buildings by CNR-GNDT et al. [11,12]

Vulnerability factors	Class score A	B	C	Weight
1. Organization of earthquake resistant system	0	1	2	4
2. Earthquake resistant system's quality	0	1	2	1
3. Conventional strength capacity	−1	0	1	1
4. Building location and foundations	0	1	2	1
5. Horizontal floor diaphragms' presence	0	1	2	1
6. Building plant configuration	0	1	2	1
7. Building elevation configuration	0	1	2	2
8. Critical elements connection	0	1	2	1
9. Type of roof	0	1	2	1
10. Non-structural elements	0	1	2	1
11. State of preservation	0	1	2	2

structural system, geometric aspects of the structure, building's state of preservation, etc. These characteristics are quantified as parameters and are evaluated considering eleven factors, and a class, score and *weight factor* is assigned to each one. There are four classes for stone masonry buildings: A, B, C and D (Table 4.1); and three for reinforced concrete buildings: A, B and C (Table 4.2).

The proposal method for reinforced concrete buildings was originally not as good as the method for non-reinforced masonry buildings. For this reason, a study by the CNR Istituto di Ricerca Sul Rischio Sismico proposed a new score for each class and weight factor value; these are indicated in Table 4.3. The weight factors tried to recognize the degree of importance of the vulnerability factors in the structural resistance of the building. The assignment of the corresponding class is done according to Table 4.4.

The *normalized index of vulnerability* (I_v) is finally obtained as the weighted sum of the product of the class score of each vulnerability factor by its corresponding weight factor. In order to facilitate the comparison between buildings in a sample, the index is normalized dividing by the maximum value that can be obtained from Eq. (4.1) in the case of

Table 4.4
Assignment of classes

Unreinforced masonry buildings		Reinforced concrete buildings	
Classes	Description	Classes	Description
A	Designed to resist lateral force	A	Good
B	Good, without seismic design	B	Regular
C	Regular	C	Bad
D	Bad		

unreinforced masonry buildings, and Eq. (4.2) in reinforced concrete buildings. When the building is very susceptible to collapse during an earthquake, the maximum normalized value is 1.0; this value tends to be 0.0 when the structure is free of damage.

$$I_v \text{ NORMALIZED} = \frac{\sum_{j=1}^{11} \left(\text{Weight of factor}_j \cdot \text{Class score of factor}_j\right)}{382.5} \quad (4.1)$$

$$I_v \text{ NORMALIZED} = \frac{\sum_{j=1}^{11} \left(\text{Weight of factor}_j \cdot \text{Class score of factor}_j\right)}{31} \quad (4.2)$$

The *Organization of earthquake resistant system* factor has a weight factor value greater than the rest of the vulnerability factors (see Table 4.3) because this factor considers many properties of the buildings that are relevant to the seismic behaviour of the building, like the six sub-factors indicated in Table 4.6.

The *Conventional strength capacity* factor has a negative value for Class "A" (see Table 4.3) as a consequence of over-strength that could be present in the building. In this case, the nonlinear deformation incursion does not occur and the seismic response therefore remains below the elastic limit, considerably reducing the damage that the structure could suffer.

4.1. Application of CNR-GNDT method to Chilean buildings

The CNR-GNDT method was first applied in Chile by Sáez [26] on adobe houses constructed in the province of Ñuble. Aranda [4] later used the method on reinforced concrete buildings of four or more storeys in the city of Concepción. The results of this study were published by Giuliano and Aranda [16] and are summarised in Table 4.5 and Eq. (4.3).

$$I_v \text{ NORMALIZED} = \frac{\sum_{j=1}^{11} \left(\text{Weight of factor}_j \cdot \text{Class score of factor}_j\right)}{29} \quad (4.3)$$

Motivated by the possibility of using a new tool in the field of seismic vulnerability of structures, Acevedo [1] and Aguirre [3] incorporated the CNR-GNDT methodology proposed by Aranda [4] into a study of seismic vulnerability of low buildings with reinforced concrete and masonry shear walls. The results of these studies showed the necessity for some changes to the CNR-GNDT methodology, considering the specific properties of both types of buildings.

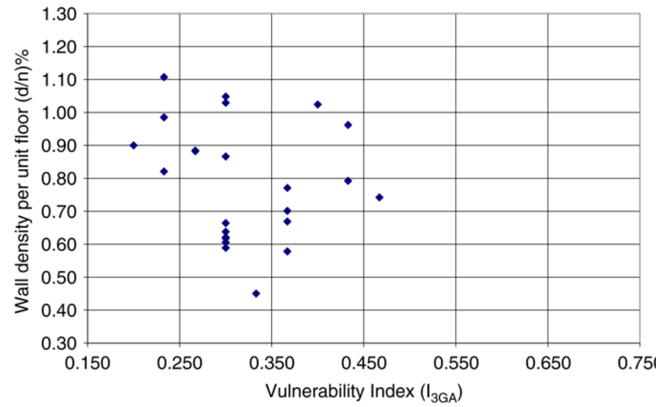

Fig. 4.1. Relationship between CNR-GNDT Index (I_v) and wall density per unit floor (d/n) in %.

Letelier [20] studied the application of the CNR-GNDT method with reinforced concrete buildings, using the buildings analyzed by Riddell et al. [25] after the March 1985 Central Chile earthquake. Instead, Gent [15] proposed a new vulnerability index for confined masonry buildings, the results of which are central to the present work.

4.2. Seismic vulnerability index for confined masonry shear wall buildings, I_{3GA}

The seismic vulnerability index for confined masonry shear wall buildings is obtained modifying the Aranda proposal (2000) and considering the ranking that results when the wall density per unit floor index is applied. The buildings studied by Acevedo [1] and Aguirre [3] were considered for this purpose. The relationship between the normalized vulnerability index (I_v) of the CNR-GNDT method [4] and the wall density per unit floor index, d/n, [18] is shown in Fig. 4.1.

Fig. 4.1 shows no clear relationship between (I_v) and d/n; this result would be considered a contradiction because the seismic vulnerability of this regular building should be related to the density of walls in the plan. In order to obtain a better relationship between (I_v) and d/n, Gent [15] modified the following vulnerability factors of the CNR-GNDT method: *Organization of earthquake resistant system, Earthquake resistant system's quality* and *Conventional strength capacity*. Modification of the seven remaining factors have been analyzed in joint form with Letelier [20] using the characteristics and available information of 41 reinforced concrete buildings. The detailed modification of these factors can be consulted in [20] and [15].

The modification process was developed by hand and does not involve any type of heuristic programming that optimises or guarantees the finding of an optimal combination. The final result is an iterative adjustment in which only the *weight factor* values are changed and the class scores proposed by Aranda [4] are not modified. The *weight factors* obtained by Gent [15]

Table 4.5
Class score and weight factor of confined masonry buildings, by Gent [15]

Vulnerability factors	Class score A	B	C	Weight [4]	Weight [15]
1. Organization of earthquake resistant system	0	1	2	4	4
2. Earthquake resistant system's quality	0	1	2	1	3
3. Conventional strength capacity	−1	0	1	1	2
4. Building location	0	1	2	1	0.75
5. Horizontal floor diaphragms' presence	0	1	2	1	0.5
6. Building plant configuration	0	1	2	1	0.5
7. Building elevation configuration	0	1	2	2	1.5
8. Type of roof	0	1	2	1	0.5
9. Non-structural elements	0	1	2	1	1
10. State of preservation	0	1	2	2	1

for confined masonry buildings are indicated in Table 4.5 and compared with the values proposed by Aranda [4].

Despite the comments, it is not forgotten that the number of physical and geometric restrictions in the programming is very high and complex, so that the adopted system appears to be better indicated for the conditions of the problem to be solved.

From a conceptual point of view, incorporating new measurement parameters tries to add a complementary evaluation for certain properties of the buildings that were not suitably considered (*Earthquake resistant system's quality* and *Conventional strength capacity*). Likewise, the reformulation of some expressions of certain parameters is due to the fact that many of these were unable to represent the properties through which they are evaluated (e.g. *Earthquake resistant system's quality* previously used the same qualification criteria as *State of preservation*). On the other hand, refining the categorization limits of certain parameters is done in order to make some ranks flexible that were previously too restrictive when assigning a class (maximum distance between resistant lines of sub-factor *Number of earthquake resistant lines* (Table 4.6) changed from 4500 mm to 6000 mm). The optimisation of some classification criteria is done in order to obtain greater efficiency and effectiveness in the methodology (*Building plant configuration*, *Building elevation configuration*, *Type of roof*, and all modified using Letelier's criterion [20]). Finally, the modification of weights corresponding to factors and sub-factors of the method is justified by the fact that the weights duly represent the incidence level of each factor within the global seismic response of the buildings (see Tables 4.5 and 4.6).

In general, the changes introduced to the methodology facilitate its calculation and use. The original formalization gave rise to double interpretations and left situations not considered.

4.2.1. *Organization of earthquake resistant system*

From this point of view, a bad building (under criteria of weighting the sub-factors of Table 4.6) must classify as class C, and a regular building, which suffers moderate seismic damage (not above Category 3 of Table 3.1), as class B. With the original methodology this does not always happen, since neither the weight of sub-factors nor the limits of categorization of class were suitable. This factor is conformed by six sub-factors; these are detailed in Table 4.6. The result of the calibration

Table 4.6
Sub-factor weights of *Organization of earthquake resistant system*

Sub-factor	Original weight	Modified weight
1. Building period	0.6	0.6
2. Building aspect ratio	0.3	0.3
3. Number of earthquake resistant lines	1.0	3.0
4. Quality of earthquake resistant lines	1.0	0.75
5. Distance between buildings	0.3	0.3
6. Torsional stiffness and torsional eccentricity	1.0	1.0

process for the weight assigned to the factor *Organization of Earthquake Resistant System* did not modify the original value, which remained at 4.0.

The major change in Table 4.6 is in the weight of the sub-factor *Number of earthquake resistant lines*, from 1.0 to 3.0. This increment is due to the fact that the density of walls is closely related to the resistance capacity and with the observed damage [18] in this type of building. Since the buildings in the sample have similar construction characteristics, the increase in the sub-factor weight assigned to the *Number of earthquake resistant lines* gives a greater role than that of the *Quality of earthquake resistant lines*, which changes from 1.0 to 0.75 (Table 4.6), because a larger number of walls (with similar characteristics) increases the over-strength and diminishes the levels of expected damage.

With respect to the classification criteria used to analyse the sub-factors of Table 4.6, only the maximum value for the distance between resistant lines changed (*Number of earthquake resistant lines*, from 4500 mm to 6000 mm). This decision was adopted because many of the buildings that suffered minor damage did not fulfil this restriction; therefore the distance suggested originally was very much conservative.

4.2.2. *Earthquake resistant system's quality*

This factor was modified with respect to the original proposal because it assigned the corresponding classes based on the year when the building was constructed, as it makes the factor *State of preservation*.

For this factor, the proposal incorporates new building properties. These are related to the number of well-confined walls and percentage of piers ("short walls") in the structure,

Table 4.7
Expected confined masonry building damage level as a function of wall density per unit floor [18] and the proposed vulnerability index I_{3GA}

Damage level	Damage category	Wall density per unit floor (d/n) %	Vulnerability index I_{3GA}
Light damage	0 and 1	$d/n > 1.15\%$	$I_{3GA} < 0.350$
Moderate damage	2	$0.85\% < d/n \leq 1.15\%$	$0.350 < I_{3GA} \leq 0.475$
Severe damage	3	$0.5\% < d/n \leq 0.85\%$	$0.475 < I_{3GA} \leq 0.600$
Grave damage	4 and 5	$d/n \leq 0.5\%$	$0.600 \leq I_{3GA}$

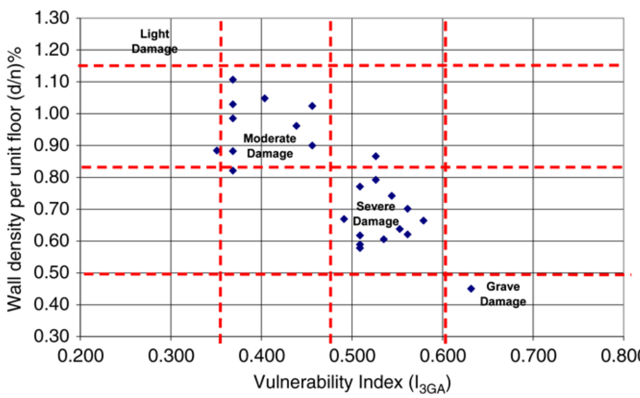

Fig. 4.2. Relationship between the vulnerability index I_{3GA} and wall density per unit floor (d/n) in %.

because they are decisive in the seismic behaviour of this type of building. The experimental tests have demonstrated that the well-confined walls have a very good deformation capacity and the presence of short walls produces a fragile failure, like the failure observed in R.C. short columns. Considering the antecedents and the results of the calibration process, the weight assigned to this factor was modified from 1.0 to 3.0.

4.2.3. Conventional strength capacity

Aguirre [3] emphasized the necessity to revise this factor because many buildings were classified as class C, as they showed good behaviour during the March 1985 Central Chile earthquake. To correct this situation, Gent [15] suggests calculating the building strength capacity in agreement with the mode of failure of a confined masonry wall [24] and modifies the weight factor from 1.0 to 2.0, as the calibration process had shown.

The final relationship between the index I_{3GA} and (d/n) is shown in Fig. 4.2. It demonstrates that an initial cluster of points without a possible interpretation (see Fig. 4.1) becomes a graph with an expected decreasing linear trend. With Fig. 4.2 and the relationship between the damage and the wall density per unit floor proposed by Astroza et al. [6], indicated in Table 4.7, it is possible to suggest the values of the proposed vulnerability index I_{3GA} associated with a different category of damage expected in the confined masonry buildings when an earthquake like the March 1985 Central Chile earthquake ($Ms = 7.8$) occurs. These values are detailed in the fourth column of Table 4.7 and are indicated by a vertical straight line in Fig. 4.2. The categories of damage correspond to those described in Table 3.1.

Fig. 4.3 was obtained from Fig. 4.2. In it, the I_{3GA} index values associated to a given damage level are indicated. In order to normalize the ordinate axis of this figure, it was accepted

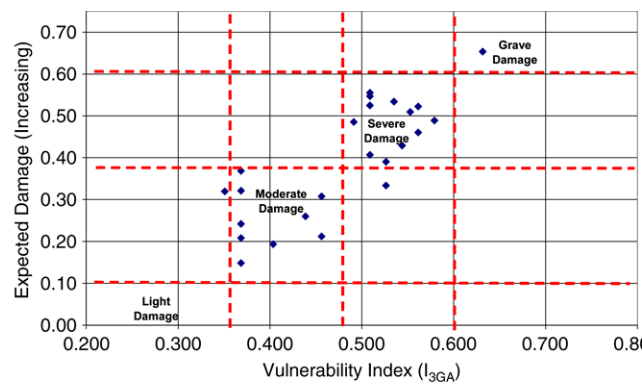

Fig. 4.3. Relationship between vulnerability I_{3GA} index and expected damage.

that seismic behaviour without damage is expected when (d/n) values are greater than 1.3%.

5. Conclusions and recommendations

The CNR-GNDT method has been modified in order to obtain a vulnerability index I_{3GA} that considers the properties of confined masonry shear wall buildings. With this objective, new values of the weight factors and sub-factors were considered in order to obtain a logical relationship between I_{3GA} and the wall density per unit floor d/n. The scope of this work is restricted to regular four storey residential buildings with R.C. slabs at floor level.

Using the experience accumulated from the affected area after the March 1985 Central Chile earthquake, $Ms = 7.8$, the I_{3GA} index values were related to certain expected damage levels. This relationship allows prognoses and predictions to be made about the behaviour of confined masonry buildings during a destructive subduction earthquake when the seismic intensity in the Mercalli Modified scale is IMM = VIII. According to the results, when the I_{3GA} index value is greater than 0.5 (deterministic procedure) the damage level should be severe.

In order to detect the zones with higher or lower seismic risk within the urban area of one city, the methods will be applied in the city of Concepcion (Chile) taking into account the vulnerability of modern constructions with seismic design, reinforced concrete and confined masonry shear wall buildings. These results will allow emergency plans to be drawn up when an offshore epicentre earthquake occurs, similar to the March 1985 Central Chile earthquake.

References

[1] Acevedo C. Estudio de la Vulnerabilidad Sísmica de los Edificios de Departamentos de la Comuna de Pudahuel (parte 1), Memoria para optar al Título de Ingeniero Civil. Santiago (Chile): Departamento de Ingeniería Civil, Universidad de Chile; 2002.

[2] Aguiar R. Sistema de Computación CEINCI3 para Evaluar Daño Sísmico en los Países Bolivarianos. Quito (Ecuador): Centro de Investigaciones Científicas Escuela Politécnica del Ejército; 2002.

[3] Aguirre A. Estudio de la Vulnerabilidad Sísmica de los Edificios de Departamentos de la Comuna de Pudahuel (parte 2), Memoria para optar al Título de Ingeniero Civil. Santiago (Chile): Departamento de Ingeniería Civil, Universidad de Chile; 2002.

[4] Aranda P. Índice Cualitativo de Vulnerabilidad Sísmica Aplicada a Edificios en Concepción, Memoria para optar al Título de Ingeniero Civil. Concepción (Chile): Departamento de Ingeniería Civil, Universidad de Concepción; 2000.

[5] Arezzo P, Bernardini A, Gori R, Muneratti E, Parisi O, Paggiarin C, Zuccaro G. Vulnerability and probability of collapse for classes of masonry buildings. In: Faccioli E, Pessina V, editors. Cap. 9.5, Volume the catania project: Earthquake damage scenarios for a high risk area in the mediterranean. CNR – Gruppo Nazionale per la Difesa dai Terremoti – Roma; 2000. p. 146–59.

[6] Astroza M, Moroni MO, Küpfer M. Calificación Sísmica de Edificios de Albañilería de Ladrillos Confinados con Elementos de Hormigón Armado. 26as Jornadas Sudamericanas de Ingeniería Estructural. Uruguay; vol 1. 1993. p. 327–38.

[7] Barbat A, Caicedo C, Canas J, Aguiar R. Vulnerabilidad Sísmica de Edificios, Monografías de Ingeniería Sísmica. Barcelona (España): Centro Internacional de Métodos Numéricos en Ingeniería; 1994.

[8] Bernardini A, Gori R, Modena C. Application of coupled analytical models and experimental knowledge to seismic vulnerability analyses of masonry buildings. In: Koridze A, editor. Earthquake damage evaluation and vulnerability analysis of building structures. INEEC series on engineering aspects of earthquake phenomena, Oxon (UK): Omega Scientific; 1990.

[9] Bernardini A, Gori R, Modena C. Fuzzy measures in the knowledge based diagnosis of seismic vulnerability of masonry buildings. In: Lin JK, editor. Probabilistic mechanics and structural and geotechnical reliability. Proceedings of sixth speciality conference, New York: ASCE; 1992.

[10] Bernardini A, Gori R, Modena C. A knowledge based survey of masonry buildings for seismic vulnerability evaluations. In: Zonno, Ducarme, editors. Proceedings of the workshop application of artificial intelligence, techniques in seismology and engineering seismology, Walferdange. Cahiers du centre européen de géodynamique et de séismologie, vol. 6. 1992. Impr. J. Beffort, Luxembourg.

[11] Consiglio Nazionale delle Ricerche (CNR), Gruppo Nazionale per la Difesa Dai Terremoti (GNDT). Rilevamento dell'Esposizione e Vulnerabilità Sismica degli Edifici, Appendice No 1 alla Pubblicazione "Rischio Sismico di Edifici Pubblici Parte I – Aspetti Metodologici", Roma, Italia. 1993.

[12] Consiglio Nazionale delle Ricerche (CNR), Gruppo Nazionale per la Difesa Dai Terremoti (GNDT). Rilevamento della Vulnerabilità degli Edifici in Muratura, Appendice No 2 alla Pubblicazione "Rischio Sismico di Edifici Pubblici Parte I – Aspetti Metodologici", Roma, Italia. 1993.

[13] Murty CVR, Brzev S, Faison H, Comartin CD, Irfanoglu A. AT RISK: The seismic performance of reinforced concrete frame buildings with masonry infill walls. First Edition Earthquake Engineering Research Institute; 2006. Publication Number WHE-2006-03.

[14] FEMA 154. Rapid visual screening of buildings for potential seismic hazards: A handbook. Earthquake hazards reduction series, vol. 41. USA: Federal Emergency Management Agency; 1988.

[15] Gent K. Calibración del Índice de Vulnerabilidad del GNDT para Estructuras de Albañilería Confinada, Memoria para optar al Título de Ingeniero Civil. Concepción (Chile): Departamento de Ingeniería Civil, Universidad de Concepción; 2003.

[16] Giuliano M, Aranda P. Índice Cualitativo de Vulnerabilidad Símica Aplicado a Edificios en Concepción, 8as Jornadas Chilenas de Sismología e Ingeniería Antisísmica, Viña del Mar. Chile. 2002.

[17] Instituto Nacional de Normalización (INN). NCh 2123.Of.97: Albañilería Confinada – Requisitos de Diseño y Cálculo. Chile. 1997.

[18] Küpfer M. Evaluaciones de las Disposiciones de Anteproyecto de Norma de Albañilería Confinada NCh 2123.c.90, Memoria para optar al Título de Ingeniero Civil, Santiago (Chile): Departamento de Ingeniería Civil, Universidad de Chile; 1993.

[19] Kuroiwa J. Reducción de desastres. Viviendo en armonía con la naturaleza, Primera Edición, Lima, Perú. 2002.

[20] Letelier V. Calibración del Índice de Vulnerabilidad Sísmica del GNDT para Edificios de Hormigón Armado, Memoria para optar al Título de Ingeniero Civil, Concepción (Chile): Departamento de Ingeniería Civil, Universidad de Concepción; 2003.

[21] Meli R. Diseño Sísmico de Edificios de Muros de Mampostería; la Práctica Actual y el Comportamiento Observado, Simposium Internacional: Seguridad Sísmica en la Vivienda Económica. 1991. p. 398–423.

[22] Modena C, Bernardini A, Gori R. A research on the seismic vulnerability of existing masonry buildings. In: Proceedings of CIB W23 — Wall structures. 1988.

[23] Moya A. Estudio de los daños del terremoto de Chillán de 1939, Memoria para optar al Título de Ingeniero Civil. Chile: Departamento de Ingeniería Civil, Universidad de Chile; 2002.

[24] Pineda P. Modos de Falla en Viviendas de Albañilería Confinada, Memoria para optar al Título de Ingeniero Civil. Concepción (Chile): Departamento de Ingeniería Civil, Universidad de Concepción; 2000.

[25] Riddell R, Wood SL, De la Llera JC. Características Estructurales y Estadísticas de Daños del Inventario de Edificios de Viña del Mar durante el Sismo de Marzo de 1985. Flores R, editor. Capítulo 12 en Ingeniería Sísmica, El Caso del Sismo del 3 de Marzo de 1985. Santiago (Chile): Instituto de Ingenieros de Chile y Editorial Hachette; 1993. p. 205–31.

[26] Sáez S. Vulnerabilidad Sísmica de Viviendas de Adobe, Memoria para optar al Título de Ingeniero Civil. Concepción (Chile): Departamento de Ingeniería Civil, Universidad de Concepción; 1999.

[27] TMS. Performance of Masonry Structures in the Northridge, California Earthquake of January 17, 1994. USA: The Masonry Society; June 1994.