# SEISMIC RISK REDUCTION MEASURES OF A VULNERABLE URBAN INFORMAL SETTLEMENT IN MÉRIDA, VENEZUELA. COST-BENEFIT ANALYSIS

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## ABSTRACT

A seismic risk assessment of an informal settlement (in Mérida, Venezuela) is presented. IVIM and NVIM methodologies are used. NVIM concludes that the buildings are vulnerable. A more detailed evaluation is performed with IVIM; the required knowledge is acquired by understanding the seismic behavior of the buildings and by code-type analyses. This output allows estimating the expected damage for selected hazard scenarios. Retrofit solutions are proposed. NVIM is used for re-evaluating the vulnerability and for determining the expected damage and number of casualties for the scenario events. A Cost-Benefit analysis is presented. This research may be useful for close situations.

**KEYWORDS:** Vulnerability analysis, Risk assessment, Risk mitigation, Non-engineered buildings, Venezuela.

## 1. INTRODUCTION

Highly populated informal settlements are increasingly spread along most of the big cities in South America and other developing regions. They contain mostly houses, both self-constructed and non-engineered; since they have rather poor quality and neither account for any code prescription nor earthquake-resistant oriented design or detailing, high vulnerability to earthquakes is expected. Moreover, these settlements frequently occupy zones with high seismic hazard; in addition, landslides and liquefaction may occur. Hence, hazard and vulnerability generate an important seismic risk (ISDR, 2001). The purpose of this research is to perform a detailed study on an informal settlement (“La Milagrosa”) located in Mérida, Venezuela. The study consists of the following steps: (1) seismic hazard analysis including microzonation, (2) building stock classification, four representative prototypes are identified, (3) code-based seismic performance analysis of these prototype buildings, (4) seismic strengthening measures (including buildings non represented by the prototypes), (5) seismic vulnerability analysis (both about the existing buildings and about those retrofitted with the proposed strategy), (6) seismic damage scenarios mapping and (7) economic and social appraisal.

## 2. NON-ENGINEERED BUILDINGS IN MÉRIDA

Mérida has more than 300,000 inhabitants. Recently a number of informal settlements have spread along the city limits, sheltering about one third of the city’s population, being the most densely populated areas and concentrating the highest vulnerability and the biggest seismic hazard in some steep sites. “La Milagrosa” is chosen because of the representativeness of its buildings and the variety of slopes in the terrain ranging from near flat to steep (60%). Apart from the risk of landslides, the soil strength is not critical since the vertical stresses are rather low (both because of the lightness of the construction and the limited number of floors). 95% of the buildings are non-engineered intermediate RC frames - confined masonry walls. About 85% of the buildings have rectangular plan configuration; the average surface is about 90 m$^2$ and the aspect ratio is 1:3 where the smallest side is the front access from the street/stairway/path. The main occupancy is housing; usually a family composed by near five members (parents and three children) occupies a single storey. Once new families arise, additional floors are added thus constituting a typical vertical growth pattern usual in most of the informal urban settlements worldwide. Most of the buildings have 1 (32%), 2 (52%) and 3 (16%) levels; only such cases are considered. Close neighboring constructions, where most of the gaps are less than 4 cm, may produce pounding, becoming critical in sloped sites, as the slabs are not horizontally aligned.

## 3. SEISMIC HAZARD IN MÉRIDA

The most important seismogenic source in western Venezuela is the Boconó Fault (Pérez, 1998), which affects Mérida in its path through the Andean Mountain range. A non-zonified probabilistic analysis is performed (Castillo, 2006) to obtain the annual exceedance probabilities for events corresponding to the European Macroseismic Scale intensity degrees $I$ = VIII and $I$ = IX. The expected horizontal PGAs are estimated using an attenuation law for western Venezuela (Arggawal, 1981). These results are shown in Table 1.

**Table 1. Scenario Earthquakes for risk analysis**

| Intensity (EMS) | Magnitude (Ms) | Maximum Acceleration (g) | Annual Exceedance Probability | Return Period (Years) |
|---|---|---|---|---|
| VIII | 6.4 | 0.1422 | 0.0117 | 86 |
| IX | 6.8 | 0.1646 | 0.0012 | 831 |

## 4. STRUCTURAL DESCRIPTION OF THE CONSIDERED BUILDINGS

### *4.1. General description*

Figure 1 shows sketches of the construction. The main features of the buildings are described next.

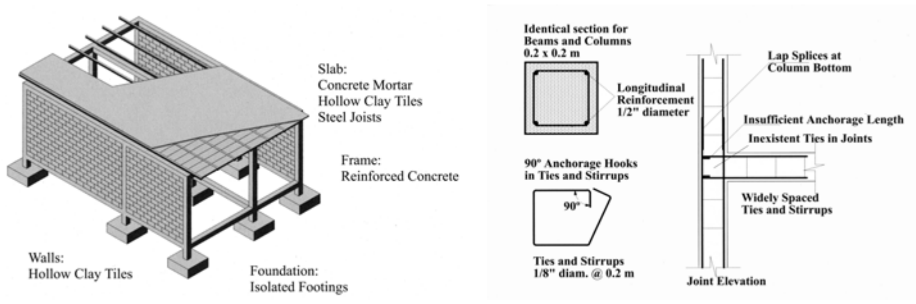


Figure 1. Structural characteristics of the houses in the “La Milagrosa” settlement

- **Foundation**. The posts are supported by isolated superficial square RC footings (averaging 0.80 to 1 m). The walls are erected without foundation; at most, a 20 × 20 cm$^2$ RC tie beam (connecting the footings).
- **Posts and ties**. These elements constitute a kind of light two-way RC frame (3-D), with average square section in beams (**ties**) and columns (**posts**) 20 × 20 cm$^2$. The beam-column connections do not have special detailing, without stirrups; however, the longitudinal rebars of the beams have hooked endings. The compressive strength of the concrete has been measured with a Schmidt Hammer; the characteristic value is near $f_{ck}$ = 10 MPa. The yielding point of the steel reinforcement is conservatively estimated as $f_{yk}$ = 140 MPa (the cheapest steel in the Venezuelan market). In some cases the alignment of the joined members is poor.
- **Walls**. The supporting walls are built after the posts in running (stretcher) bond, without reinforcement and using low-quality mortar. The bricks are hollow and not intended for structural use, their length/width/height are 25/20/15 cm; the cells are disposed horizontally. The apparent unit weight is $\gamma_w$ = 12 kN/m$^3$. The characteristic values of the shear and compressive strengths are conservatively estimated as $f_{wk}$ = 0.08 MPa and $f_{wck}$ = 0.35 MPa, respectively (IAEE, 2001). The friction coefficient is taken as $\mu$ = 0.4 (ENV-1996, 1996). The cladding walls run around the entire perimeter; openings are produced only at front and back sides. At the front of the first floor there are usually one window and one door; at the rear there are two windows. In upper levels, for both the front and rear, the openings are two windows (or one door and one window if there is a balcony). Not all the partition walls are aligned with the frame; moreover they are not always vertically coincident. The walls which are under slabs are termed in the following “topped” since the beams provide a certain confinement, while those under roofs are termed “untopped” as the roof is weak, untied and does not provides any confinement.
- **Slabs**. The slabs are built with I-shaped beams (IPN, 80 mm in height) that are arranged parallel to the longitudinal axis of the building and are supported by the transverse RC ties. The inter-axial spacing is

around 800 mm as allowing placing hollow clay blocks with width/longitude/height 200/800/60 mm. There is a topping concrete compressive layer (30 to 40 mm high) reinforced with a steel welded mesh (diameter 3.175 mm @ 20 cm). The concrete compressive strength is estimated as $f_{ck}$ = 10 MPa. The yielding point of the steel of the beams is $f_{yk}$ = 250 MPa. It is assumed that the steel of the welded mesh is the cheapest available ($f_{yk}$ = 140 MPa). The self-weight of this slab is estimated as 1.832 kN/m$^2$. Usually, the first level has a front cantilever ranging from 700 to 900 mm; it either supports masonry cladding walls or balconies.
- **Stairs**. The stairs connecting the levels may be either inside the building (parallel to the longitudinal axis) or outside it. They are made of steel or reinforced concrete (See Figure 2).
- **Roofs**. The roofing consists of light metal sheathing (usually zinc) over metallic beams (I-shaped or hollow rectangular); the roofs are not well fixed to the posts and walls. The self-weight is estimated as 0.4 kN/m$^2$.
- **Quality**. The general level of quality, yet rather poor, is better than expectable. Some deficiencies: broken masonry units and discontinuities in the walls, bonding mortar not resisting scratching, unaligned horizontal running of the blocks, light roofs not properly tied, shrinkage cracks in floors, cavities and erosions in the posts and ties (with irregularities due to framework positioning) and insufficient reinforcement cover (even some bars are visible). In some cases, soil erosion surrounding the foundations is observed in steep sites.
- **Pathologies**. No relevant pathologies are observed.

Figure 2 shows some representative pictures of the houses in the "La Milagrosa" settlement.

Figure 2. Buildings in the "La Milagrosa" settlement

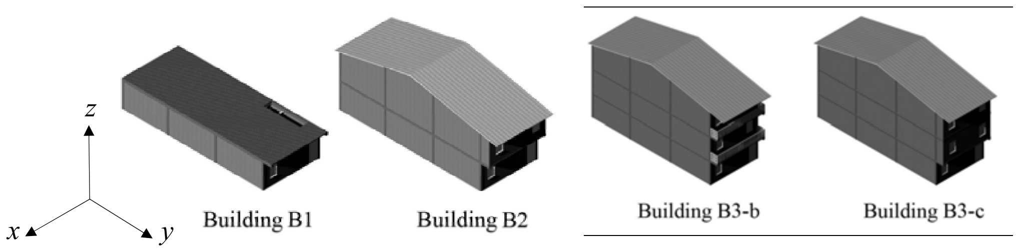


Figure 3. Prototype buildings

### *4.2. Prototype buildings*

Four prototype buildings (see Figure 3) have been selected to represent most of the houses in "La Milagrosa":

- **B1**. One storey, topped with a slab.
- **B2**. Two stories, topped with a light roof. The second floor might have either a cantilever or a balcony.
- **B3-b**. Three stories, topped with a light roof. The second and third floors have a balcony.
- **B3-c**. Three stories, topped with a light roof. The second and third floors have a cantilever.

The average height per floor is 2.80 m. It is remarkable that, at building B3-c, the eccentricity among the centers of stiffness and of gravity does not affect significantly the torsional seismic behavior.

## 5. SEISMIC ANALYSIS OF THE CONSIDERED BUILDINGS

### *5.1. Behavior under gravity loads*

The live load for the slabs is chosen as $L$ = 1.5 kN/m$^2$, for the light roofs (B2 and B3) is zero while for the roof of B1 is $L$ = 1 kN/m$^2$. The safety factors for dead and live loads are taken as 1.4 and 1.7, respectively.

- **Posts and ties**. The structural analysis of the transversal frames (those supporting the secondary beams as shown in Figure 1) show that they lack of capacity to resist the weight of the building, even under serviceability conditions: (1) the greatest demanding bending moment is near 15 times bigger than the resisting one, (2) the greatest demanding shear force is about 8 times bigger than the resisting one, (3) the transversal reinforcement is unreliable since the separation between adjacent stirrups is bigger than the effective depth, the diameter is too small, the steel yielding point is unclear and the stirrups are not well closed and (4) the overlapping length is smaller than the minimum value established in (ACI-318, 2002) and is located usually near the bases of the columns. Therefore the vertical loads are resisted by the walls. Consequently, these buildings have to be considered as "intermediate RC frames – confined masonry walls".
- **Walls**. The walls must carry most of the load; even neglecting the contribution of the posts, the assumed compression strength of the masonry ($f_{wck}$ = 0.35 MPa) is sufficient. Even for buildings B3, the average stress is $\sigma_{cw}$ = 0.32 MPa, which is smaller than $f_{wck}$. However, the addition of another plant is dangerous.
- **Slabs**. The linear elastic structural analysis of the steel beams (Figure 1) shows that, if the contribution of the upper concrete is neglected, the maximum normal stress largely exceeds the yielding point. Hence, it is required to consider the cooperation of such layer. However, as there are no shear connectors, it is unreliable to analyze each beam-topping assembly as a composite single member. In any case, the slabs are clearly unsafe according to regular standards. It is remarkable that the design manual (Sidetur, 2004) recommends span length up to 3.45 m and requires a 6 cm deep top layer cast with higher strength concrete ($f_{ck}$ = 25 MPa).

Hence, demands exceed resistances. However, relevant pathologies have been neither observed nor reported. This disagreement can be explained by the difference between serviceability and ultimate conditions, by the discrepancy among actual live loads and the considered ones and by the assumed conservative simplifications.

### *5.2. Structural models for horizontal loads*

The horizontal seismic behavior of the prototype buildings is described by lumped mass models. Since the walls are significantly stiffer than the posts, they take most of the horizontal forces and, therefore, the position of the center of rigidity is rather governed by the walls distribution (yet accounting for the openings); the buildings have plan symmetry as the eccentricities between the centers of mass and of rigidity do not exceed 5%. Hence, the behavior in each horizontal direction is described with a model with one degree of freedom per floor.

### *5.3. Equivalent seismic forces*

The equivalent demanding forces are determined (Castillo, 2006) according to the Venezuelan seismic design code (MINDUR and FUNVISIS, 1998). The resulting forces are shown in Table 2.

**Table 2. Lateral demanding forces**

| Prototype Building | $F_1$ (kN) | $F_2$ (kN) | $F_3$ (kN) |
|---|---|---|---|
| B1 | 309 | - | - |
| B2 | 335 | 227 | - |
| B3-b and B3-c | 237 | 475 | 241 |

### *5.3. Expected seismic performance*

Several (global or local) failure modes are possible (Paulay and Priestley, 1992):

- Shear failure of the topped walls accompanied by shear or bending failure of the posts. The failure of the masonry goes along horizontal mortar courses, generally at mid-height. The initial resistance of the walls is determined by Mohr-Coulomb models (City University of London, 2005) with the aforementioned values of shear stress strength and friction coefficient; for cyclic behavior, only the friction term is reliable. The effect of openings is represented by a reduction in the wall length. The shear resistance of the posts is obtained according the seismic ACI criterion (ACI 318, 2002) neglecting the transversal steel contribution since the excessive separation among consecutive stirrups allows the formation of shear cracks, Figure 1. The strength

of the posts to collapse by formation of plastic hinges is determined by push-over analysis; since enough rotation capacity in the hinges can not be assumed (mainly because of the lack of transversal confinement), it is conservatively assumed that the posts collapse after the formation of the first set of plastic hinges.
- Diagonal strut compression failure of the topped walls (those coplanar with the posts and ties, since otherwise there are no adjacent vertical supporting elements able to provide vertical compression forces) accompanied by shear or bending failure of the confining elements. The strength is determined by classical ties and struts models (Paulay and Priestley, 1992); in walls with openings the diagonals are more vertical. This resistance is upper bounded by the one of the posts to shear.
- Collapse of propless elements. Due to the vertical seismic action, unsupported elements (vertically discontinuous walls and cantilevered walls) can collapse. This risk is serious as the supporting ties and slabs are demanded by the gravity loads beyond their capacities. Moreover, the walls carry most of the weight.
- Out-of-plane failure of the untopped walls (supporting light roofs). The resistance is low reliable and hard to estimate given the poor quality of the mortar and the lack of upper collar beams and of reliable ties.
- Detachment of the roofs from the supporting elements. The roofs, are not rigidly connected to the walls and posts and, hence, are in serious risk (hard to estimate) of falling. This might have fatal consequences.

The global resistance in each direction is determined for the most critical combination of these mechanisms. Table 3 displays the results for building B1; demands exceed strengths. The resistance in the transversal direction ($x$, see Figure 3) is significantly smaller than in the longitudinal one in spite the walls are longer and have no openings; it is due to the premature shear failure of the posts because of the diagonal compression struts formed in the walls. For buildings B2 and B3, the demanding forces are bigger (Table 2) while neither the strengths nor the critical failure modes change significantly. The cantilevers are in serious risk because of the vertical input.

**Table 3. Seismic behavior of building B1**

| Direction | Failure mode | Strength (kN) | Demand (kN) |
|---|---|---|---|
| $x$ (transversal) | Shear of the cladding and partitioning walls (brittle) | 293 | 309 |
| $y$ (longitudinal) | Shear of the posts and of partitioning walls (brittle) | 174 | 309 |

## 6. A PROPOSAL FOR SEISMIC STRENGTHENING

### *6.1. Introductory remarks*

This section presents feasible solutions to reduce the seismic vulnerability of the non-engineered constructions in “La Milagrosa”. It is assumed that any intervention, even made by the owners, be enforced and technically supervised; i.e., instead of non-engineered construction, it should be rather termed *engineered-self-construction*. Only simple and inexpensive solutions are proposed, the codes are not fulfilled and, hence, a complete protection against earthquakes may not be guaranteed. The main seismic deficiencies lie in two broad categories: resisting elements and configuration; this last applies mainly for the buildings not represented by the four prototypes.

### *6.2. Strengthening of the resisting elements*

- The buildings are mainly supported by the cladding and partitioning walls; hence, they should not be partially or totally removed. New openings should be made carefully using temporary props vertically continuous down to foundation and placing lintels and jambs. If there are partitioning walls which are not vertically continuous down to the foundation, new walls should be erected to guarantee such continuity. All the walls which are not coplanar with the posts and ties should be moved to coplanar positions. Particularly, those upper level cladding front walls built over the edge of cantilevers ought to be relocated back.
- The transversal cladding and partitioning walls ($x$ direction, see Figure 3) should be coated with of RC (City University of London, 2005). Benefits: to allow the development of the full strength capacity of the existing walls and to provide additional lateral strength; this last can be estimated as 90% of the resistance of the horizontal steel bars and, because of the dowel effect, 20% of the vertical ones (City University of London, 2005). The demands (Table 2) can be easily fulfilled. Particular attention should be paid to corners or toes since are zones with stress concentrations for collapse mechanisms. The not cantilevered outer walls should be chosen preferably for strengthening, since it would both keep plan symmetry and provide higher torsion strength. The detailing should guarantee an even contact and a proper anchorage with the surrounding supporting elements (slabs, ties, posts) for a smooth load transfer. For the first floor walls, some foundations

are required to get enough confining; Figure 4 (left) shows a sample solution for a frequent situation.

- The longitudinal walls ($y$ direction, see Figure 3) can be strengthened by either (1) placing additional horizontal hooked steel reinforcement bars in the bed joints (see Figure 4 -right-) (Valluzzi, Binda and Modena, 2005) or (2) lining one or both sides of the wall (Figure 4 -right-) with anchored layers of reinforced concrete. The main benefit is to increase the strength to all the failure modes: diagonal compression, horizontal shear and out-of-plane. Since these walls support a relevant part of the weight of the building, this operation should be performed carefully; props (continuous down to the foundation) are required.
- To top all the untopped walls with collar beams (ties) connected to the posts. These ties can have the same cross section and reinforcement as the other existing frame members and are only intended to tie the wall and to support a light and un-detachable roof; consequently, under no circumstances any floor can be built over.
- If possible, the resistance of the steel secondary beams should be increased. This measure is oriented to increase the strength for gravity loads but it yields also some benefits for the vertical components of the seismic inputs, mainly if the steel beams support uncoplanar unpropped walls.

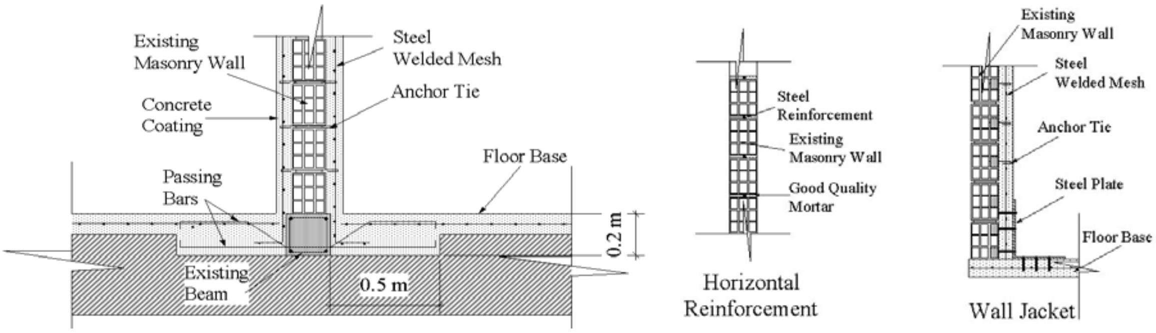


Figure 4. Transversal wall jacketing (left ) and longitudinal wall strengthening (right)

### *6.3. Solutions to configuration problems*

- The asymmetric buildings can be re-symmetrized by adding infill walls (which are coplanar with the posts and ties) or by closing (totally or partially) some openings.
- In steep sites, the front space between poles should be filled with (well made) masonry walls. This would reduce the high asymmetry generated by the restraint exerted by the rear ground supports.

### *6.4. Additional measures*

- Do not erect additional floors. It applies even to single story buildings.
- To ease the pounding between adjacent buildings. If the slabs are unaligned (typically, in steep sites), some vertical, stiff and resistant elements (steel or timber bars) should be placed in between the two buildings to reduce the most damaging effects of pounding. If the slabs are aligned, some horizontal resistant yet absorbing elements (e.g. timber pads) should be placed in between any two adjoining slabs.
- To avoid roofing with heavy materials (tiles, concrete blocks, massive steel members, among others); conversely, the use of isolating zinc sheathing supported by light steel or timber elements is encouraged.
- To fix the heavy and tall furniture and appliances to the floor or to the posts.
- To avoid detachable roof parts. Heavy unanchored elements used to avoid sheathing uplift (bricks, rock, tires, among others), are not advisable because of the risk of fall and of the added mass.
- Excretal waters should be drained to the public sewage system to avoid soil problems due to local excess of water, especially in the lower parts of the premises.

## 7. VULNERABILITY ANALYSIS

Two approaches are considered to assess the vulnerability and the expected damage: the well known Italian Vulnerability Index Method (IVIM) and a recent modification that uses the EMS-98 (Giovinazzi and Lagomarsino, 2004; Giovinazzi, 2005) referred as New Vulnerability Index Method (NVIM); it is used to obtain the expected damage for selected hazard scenarios. The building fragility is defined by an index ranging from 0 to 1 (from least to most vulnerable). The damage is evaluated by vulnerability functions which, for a given hazard scenario and for a given vulnerability index, allow predicting expected damage; it is quantified by a central damage factor ranging usually also between 0 and 1. IVIM considers two main typologies: masonry and reinforced concrete; the buildings in "La Milagrosa" can not be classified strictly under any of them; are treated as reinforced concrete because the methodology for masonry is inadequate. IVIM is a score methodology that punctuates buildings based in eleven parameters (GNDT, 2001); each parameter is qualified by coefficient $K_i$ and is weighted by $W_i$ factor. Coefficients $K_i$ are scored for three levels: *A* (no added vulnerability), *B* (moderate increment of vulnerability) and *C* (highest vulnerability growth). IVIM offers a complete assessment suite for the major observed deficiencies.

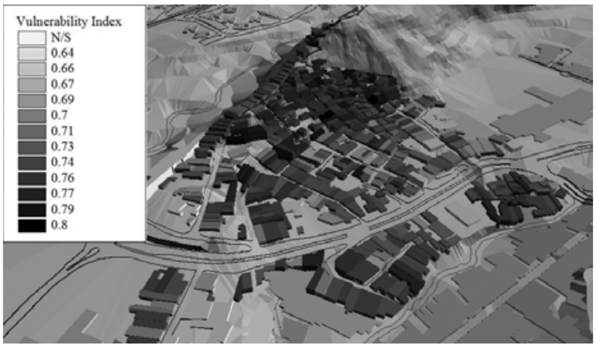


Figure 5. Vulnerability index distribution inside the settlement. N/S means Not Studied

Vulnerability functions can be built either from observed damage or by numerical simulation (Yépez, 1996) but, since there are few damage data and the IVIM vulnerability functions are calibrated for Europe, NVIM has been preferred because of its generality (Giovinazzi, and Lagomarsino, 2004; Milutinovic and Trendafiloski, 2003).

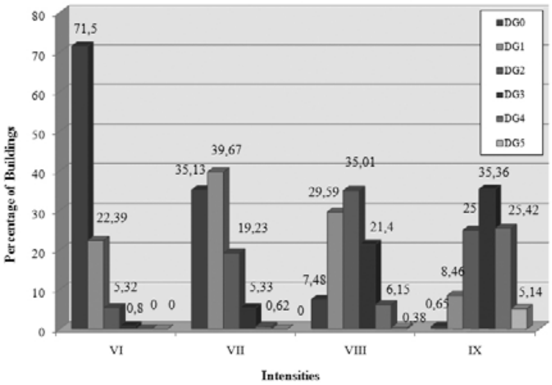


Figure 6. Damage probability distribution for several intensities

A vulnerability and damage analysis of the un-strengthened buildings in "La Milagrosa" using IVIM and NVIM is performed. NVIM is used to predict expected damage. The information required for scoring the twelve parameters required by IVIM has been obtained from comprehensive site inspection (Section 4) and from code-type structural analysis (Section 5). Figure 5 shows the geographical distribution of the vulnerability indices inside the settlement. The most probable values of the NVIM vulnerability index, have been used to predict expected damage for several hazard scenarios. Figure 6 shows the obtained damage distributions.

## 8. ECONOMICAL APPRAISAL

**Table 4. Benefits of the strengthening**

| | Unstrengthened buildings | | Strengthened buildings | |
|---|---|---|---|---|
| Input intensity | Economical losses (Millions of US$) | Death casualties | Economical losses (Millions of US$) | Death casualties |
| $I$ = VIII | 1.34 | 45 | 0.04 | 2 |
| $I$ = IX | 5.36 | 275 | 0.39 | 10 |

To estimate the benefits, the distribution of expected damage and the number of casualties is assessed for the actual state and for the strengthened buildings. From the damage state probabilities, NVIM (Milutinovic and Trendafiloski, 2003) allows estimating the number of injured and deceased people, as well as economical losses. Victims are estimated as (Vacareanu et al., 2004). Table 4 summarizes the results. The cost of the proposed strengthening for the whole settlement is roughly estimated as 1,040,000 US$ while to rebuild the houses would cost almost 19,000,000 US$. The workforce cost is not included as will be supplied by the owners (mainly manual tasks) and by the government (mainly technical consultancy and supervision). It is remarkable that to incorporate these issues into the costs would widen significantly the gap among them.

## 9. CONCLUSIONS

- The buildings in "La Milagrosa" are highly vulnerable. This conclusion is supported by code-type analysis of four representative prototype buildings and by global vulnerability analysis.
- A rather moderate investment can provide a reasonable level of seismic safety (via self-made retrofit) for settlements like "La Milagrosa". Such amount is less than the 6% of the reconstruction cost and is smaller than the economical losses in case of an earthquake with intensity VIII. For intensity IX, 265 lives (out of 275) would be saved.
- For seismic strengthening, following a rather self-construction process might be acceptable; conversely, the enforced technical supervision is a must.

## ACKNOWLEDGEMENTS

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