Chapter 7

7   Construction and Strengthening Recommendations

The essential objective of seismic risk assessments is to provide information about the possible physical damages over the buildings and the possible losses (economical and social) generated in the degradation process. This information is useful to propose strategies for risk reduction, which mainly consist in mitigation of the possible damage inflicted upon the buildings given a certain earthquake (i.e. the reduction of seismic vulnerability of buildings).

The mitigation is oriented both to improve the standards for construction of new buildings and to seismically strengthen the existing ones (including those under construction, except perhaps the just started ones). Both approaches are usually applied simultaneously; however, their costs per building are considerably different, as the first strategy deals with new constructions and its incremental cost is significantly small if compared to that required for existing buildings retrofit. Moreover, this last approach not only confronts the difficulty to reliably assess the vulnerability of the buildings detecting the most relevant deficiencies for seismic resistance, but the economical, social, cultural and technological adequacy of the proposed solutions at the expected level of vulnerability reduction.

The strengthening measures are the cornerstone of vulnerability reduction, as the number of existing buildings in any urban center (including those under construction) is many times greater than the new ones (in a reasonable period of time). It is well known that the turnover rate of the building stock (new buildings over older ones) is usually very low even in industrialized countries, where this rate may usually be around a 1% annually, finding its maximum around 8%, this means that the total number of older buildings does not diminish rapidly [Coburn and Spence, 2002].

This chapter presents a preliminary study about some feasible measures to reduce the seismic vulnerability of the non-engineered constructions in "La Milagrosa". The proposed solutions are classified in construction and strengthening recommendations; the first ones are intended for new construction while the second ones are for existing buildings. It is considered that any applied measure, even made by the owners, should be technically supervised and enforced; i.e., instead of non-engineered construction, it should be rather termed *engineered-self-construction*. An economical assessment of the benefits-investment ratio is presented for the strengthening measures. Some strategies to spread the results of this research among the involved parts (e.g. officials, local community, etc.) are also described.

7.1 Draft Construction Recommendations

A list of important construction recommendations is included next. It is intended only for new constructions. The guidelines are cheap and easy to follow (neither rare materials nor complicated techniques are suggested), in order not to increase significantly the cost and the difficulty of the construction. It is remarkable that no highly skilled workers are required in order to maintain the self-construction nature. However, the expected benefits are important. In any case, it is remarkable that these measures will not guarantee the complete fulfillment of all the existing codes as it would be virtually impossible without changing completely the construction techniques.

General description of the proposed buildings

The proposed buildings will be similar to the existing ones, since high deviations from the actual techniques are not considered as to avoid the need of training people. The most relevant innovations are:

- The number of floors is absolutely restricted to two. The construction can be progressive, i.e. the first floor is erected first and, when needed, a second floor is built over. The roof of the second storey should be light.

- The cladding and partitioning walls are founded on reinforced concrete stripes. The expected benefit is to guarantee a smooth and even transmission of vertical (for gravity loads) and horizontal (for seismic inputs) stresses to the ground. The wall foundations will additionally serve as ties between the columns footings (preventing relative movements between them during strong earthquakes).

- The walls are vertically continuous (down to foundation) and coplanar with the frames. The benefit is a better structural cooperation between walls and frames.

- The resistant quality of the walls is improved by using better mortar and workmanship. This is relevant as it has been found that the walls carry most of the weight (subsection 6.3.4) and lateral loads (subsection 6.3.5).

- The longitudinal walls are reinforced with horizontal steel bars embedded in mortar beds and anchored in the columns. This measure will significantly increase the strength to the diagonal strut compression and the out of plane failure modes; this will provide more ductility.

- Stirrups are closer, especially near the joints (as the shear forces are bigger and plastic hinges are expected); particularly it never exceeds the effective depth of the section. Benefits: to avoid premature shear failure of beams and columns, to confine the core concrete and to guarantee ductility (useful for plastic hinges).

- The strength of the cross-sections of the beams and of the columns should be increased by enlarging the depth and the reinforcement amount (and/or the steel yielding point). The beams will be 20 cm wide and 30 cm deep, while the columns will have the same section (20×30 cm^2), where the highest dimension is in the x direction (main resisting frames). Eventually, the use of square sections (30×30 cm^2) could be recommended to avoiding the possibilities of construction errors and of eccentricities in the joints. The reinforcement capacity can be increased either by enlarging the amount or by using steel with higher yielding point.

- All the upper floor walls are topped with collar beams. Benefits: more resistance to out of plane failure and better support of the roof.

- The construction process is different, as described in the following. The main innovation consists is erecting the walls before the columns and beams and to cast them against the walls. The main benefits from this strategy are to confine the walls and to increase its vertical compression (it improves the shear, diagonal compression and out of plane strength). This technique has been already used in other non-engineered houses in Mérida. Picture 18, shows a confined wall building under construction, where the technique is observed (another Barrio in Mérida).

Picture 18: Building under construction

Construction process

The construction process is described herein sequentially. It is assumed that the vertical growth is progressive (a floor is erected at a time).

- **Foundations**. To lay the reinforced concrete foundations for columns (footings) and walls (stripes). The reinforcements of the first floor columns should be left embedded (with proper anchorage) into the footings. Since the length of the commercially available bars is, at least, 6 m, it is advisable to keep their integrity and to form the reinforcement for two consecutive floors (even stirrups) without splicing; if the completion of the second floor is delayed (even for decades), the exposed bars should be protected from corrosion. In any

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case, enough length for splicing (at least 40 cm) should be left protruded from the first floor (it is remarkable that the column base is not the best position for overlapping the longitudinal reinforcement).

- **Walls**. The first floor infill walls (cladding and partitioning) are erected on the stripe foundations using a stretcher bond. In the longitudinal walls, at $h/4$, $h/2$ and $3h/4$, (h is the height of the wall) the mortar beds must be reinforced with (at least two) steel bars anchored to the columns reinforcement; if required, the height of the mortar bed could be increased to house the bars (and to protect them against corrosion). These reinforcements will constitute a kind of simplified ring beam. In the transversal walls, these reinforcements are practically useless if their continuity is interrupted by openings, and only the other ones (those situated above or below the openings) are required. The reinforcements above the openings play the role of lintels. As the bond is stretcher, the sides facing the columns are toothed; the voids can be cast (prior to the columns casting) with mortar or concrete to avoid the risk of bad filling during the main casting operation. The bond with the column can be improved by leaving protruding re-bars.

- **Moulds**. The columns will be molded with conventional wooden planks. They can be tied each other by usual techniques using conventional wires. The moulds can either cover the whole length of the column (casting in a single operation) or be shorter; this last option will ease the concrete compacting). The beams are molded in the same way as in the existing buildings.

- **Casting**. The columns are cast by conventional gravity screed. The concrete compacting can be made either by shaking the moulds (this operation is possible as they are fixed to the walls) or by striking directly in the mass with conventional steel bars. Some of the holes of the bricks will be partially filled with the fresh concrete, this will benefit the bonding. The workers should be informed that excess of water (yet improving the workability) reduces the strength.

- **Steel beams**. The structural continuity between the IPN sections and the upper concrete layer is guaranteed by welding shear connectors to the upper flange of the steel beams. The upper endings of these connectors should be flat and aligned horizontally as to provide an even support for the welded mesh. The estimated height is around 2 cm. In this way the mesh is better protected against corrosion and it can cooperate more intensely with the surrounding concrete.

- **Slabs**. The slabs will be built roughly like in the existing buildings. A bondage may be practiced between the slab and the (transverse) beams, by leaving protruding steel bars at the upper side of the beam, and also a space between the hollow clay bricks arrangement (over the beams), that will be used to fill with concrete when the topping of the slab is cast. As discussed previously, the mesh is separated 2 cm from the upper flange of the steel beams; hence the minimum total height of the concrete layer is 5 cm.

Figure 7.1, Figure 7.2 and Figure 7.3 describe first three major consecutive operations, respectively.
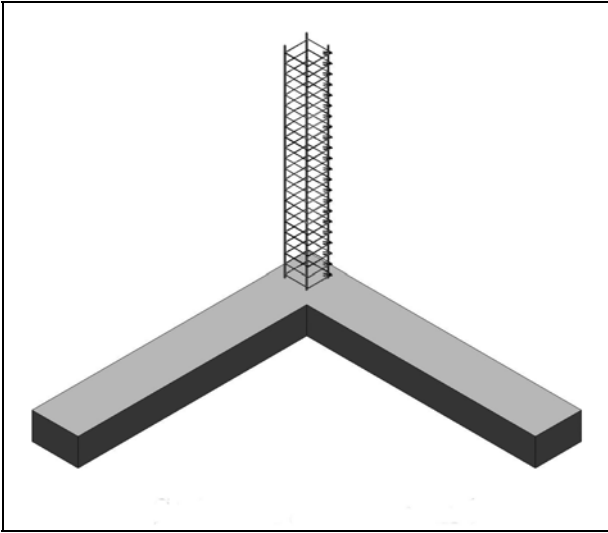
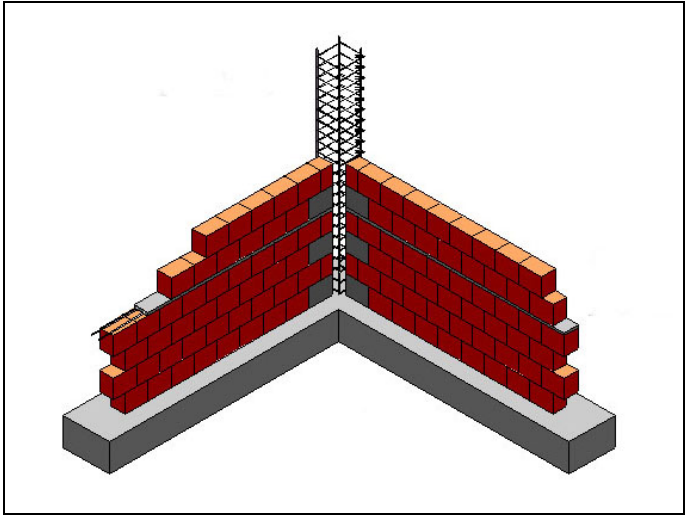**Figure 7.1: Stripe foundation and column reinforcement**
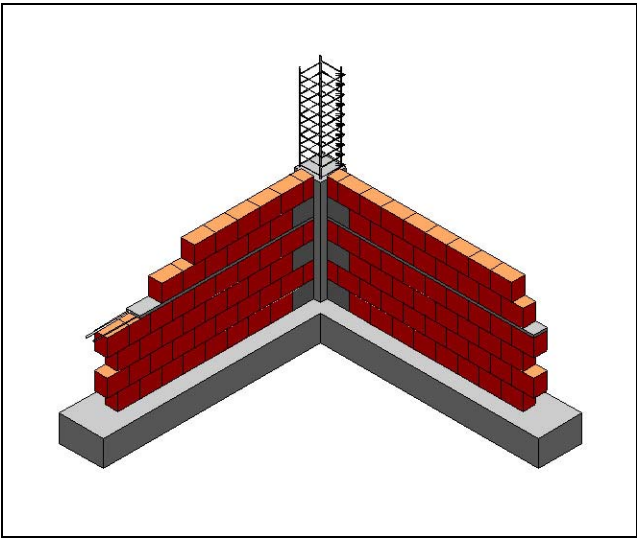

Figure 7.2: Walls

Figure 7.3: Columns

Construction details

- The excretal water should be drained to the public sewage system to avoid possible soil problems due to local excess of water (especially in the lower parts of the premises).

- The footings for the columns should be, at least, 50 cm wide (square) and 30 cm high. The (stripe) foundations of the walls should be, at least 50 cm wide and 30 cm high. It means that the whole foundation will be striped 50 cm wide and 30 cm deep.

- The reinforcement for both foundations should be, at least, a steel welded mesh 10×10 cm with 3/8" diameter. Anchoring will be obtained by proper hooking.

- Stirrups' hooks should be closed (135º). Benefits: to improve the confinement of the core concrete.

- To better anchor the beam longitudinal reinforcements in the columns, such bars should be hooked to the longitudinal bars of the columns.

- In the joints between beams and columns their axes should be coplanar (concurrent) to avoid eccentricities. However, if the sections are not square (20×30 cm^2), some minor eccentricities among the axes of beams and columns arise if the beams are aligned with the external side of the columns; this practice is not recommended but the effects (torsion in the columns, for lateral actions) will not be very important (since most of the horizontal forces will be carried directly by the slabs).

- To avoid roofing with heavy materials. Benefit: to reduce the seismic forces.

- To avoid detachable roof parts. Heavy elements used to avoid sheathing uplift (bricks, rock, tires, among others), are neither enough nor advisable (because of the risk of fall and of the added mass).

Additional recommendations

- The building is mainly supported by the walls. Hence, they should not, under any circumstances, be removed (even temporarily) or pierced significantly.

- Plan symmetry should be sought. Sources of asymmetry: steel stairs, wide openings (in external walls). The asymmetric buildings can be re-symmetrized by adding infill walls (which are coplanar with the frame) or closing openings. Benefits: to avoid torsion effects.

- In sharp or irregular construction sites, the symmetry is still more important. Since construction joints are unfeasible, stiff elements (stairs and good quality infill walls) should be placed at both ends. Benefits: to limit torsion effects.

- In steep sites, the space between poles should be filled with masonry walls (mainly in the outer frame). Benefits: to assure structural symmetry parallel to the contour lines.

- To keep enough seismic gap with contiguous buildings (mostly if slabs are misaligned, usually in steep sites). Benefit: to avoid pounding effect.

- To fix the heavy and tall furniture and appliances to the floor or to the frame. Connections to the walls should be avoided because of the risk of out of plane failure.

- To avoid cantilevered cladding walls. Balconies are acceptable as both the dead and live loads are smaller. It benefits both the behaviors for gravity loads and for the vertical component of the seismic action.

7.2 Draft Strengthening Recommendations

A preliminary list of important strengthening recommendations is included next. The guidelines are cheap and easy to follow (neither rare materials nor complicated techniques are involved), in order not to increase significantly the cost and the difficulty of the construction. It is remarkable that no highly skilled workers are required in order to maintain the self-made nature. However, the expected benefits are important. In any case, it is remarkable that these measures will not guarantee the complete fulfillment of all the existing codes as it would be virtually impossible.

7.2.1 Suggested measures

The main seismic deficiencies of the buildings in "La Milagrosa" lie in two broad categories: resisting elements and configuration. The proposed measures are described next.

Resisting elements

- The building is mainly supported by the cladding and partitioning walls. Hence, they should not, under any circumstances, be partially or totally removed. If new openings are intended, they should be made carefully; it includes the use of temporary props (vertically continuous down to foundation) and lintels and jambs (made with reinforced concrete or steel). It is not advisable to practice openings in the side walls. If there are walls which are not vertically continuous, new walls should be erected in the lower levels to guarantee such continuity. If possible, new walls should be placed in coplanar (with the frame) positions.

- The transversal walls (x direction, both cladding and infill walls) are strengthened by coating them with two (outer) layers of reinforced concrete [City University of London 2005]. The construction process consists of the following consecutive operations: (1) to pierce a number of holes (about 1 cm diameter) in the joints of the wall, (2) to insert some short (deformed) steel bars in such holes and to bend them by their ends as to form two hooks, (3) to place two welded steel meshes at both sides of the wall and to anchor them to such hooks, (4) to cast two layers of concrete (about 3 cm thick) to both sides of the wall. Benefit: to increase the strength of the wall in all the directions. For the first floor walls, some foundations are required to get enough confining as to allow the formation of the failure mechanisms (instead of soil collapse); these foundations might be constituted by lateral stripe footings connected to the concrete jacket layer. Figure 7.4 shows a sample solution (for a frequent situation). Particular attention should be paid to zones with stress concentrations for collapse mechanisms (e.g. corners or toes, see Figure 6.19, Figure 6.20, Figure 6.21 and Figure 6.22). The strengthened walls should be primarily the outer ones (cladding) as to keep plan symmetry and providing less sensitivity to accidental eccentricities. The detailing should guarantee an even contact and a proper anchorage with the surrounding supporting elements (e.g. slabs, beams, columns) as to allow a smooth and sound load transfer (even for gravity loads).
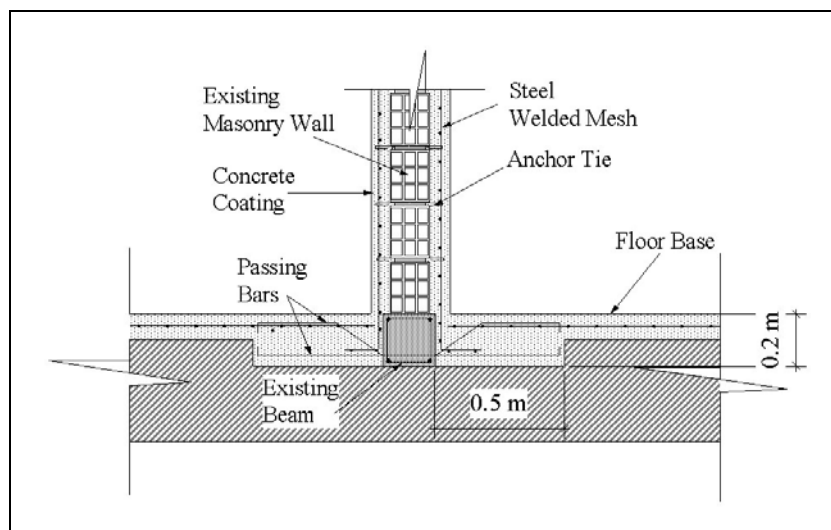
Figure 7.4: Transversal wall jacketing

- The longitudinal walls (*y* direction) are strengthened by two alternative measures: (1) placing additional horizontal (hooked) steel reinforcement bars in the bed joints (removing temporarily part of the mortar, inserting the bar and regrouting, see Figure 7.5 left) [Valluzzi, Binda and Modena 2005] and (2) lining both sides (if possible, otherwise one side would be enough) of the wall with anchored layers of reinforced concrete (as described by Figure 7.4 for transversal walls). Figure 7.5 (right) shows a sample solution for a particular case. Benefit: to increase the strength to the diagonal compression, horizontal shear and out of plane failure modes. Since these walls support part of the weight of the building, this operation should be performed carefully; props (continuous down to the foundation) are required. The first operation (additional bars in the bed joints) is not advisable for the transverse walls (*x* direction) as the diagonal compression failure mode is not possible and the vertical compressions are higher (this operation is risky as the wall is exposed to out of plane failure threat). For the first floor walls, foundations are needed and built as in the transversal walls.

- To top all the walls (with collar beams connected to the columns). These beams can have the same cross section (200×200 mm^2) and reinforcement (both longitudinal and transversal) as the other members. It should be kept in mind that these beams are only intended to tie the upper portion of the wall and to support a light and un-detachable roof; consequently, under no circumstances any floor can be built over.

- Slabs. If possible, the depth of the steel secondary beams should be increased by welding to the lower flange something similar to half IPN profile or a T section. This solution is described by Figure 7.6.

Configuration problems

- If the building is asymmetric (i.e. the centers of gravity and of rigidity are not vertically aligned), the seismic motions induce torsional effects that might generate premature failure of some walls (probably those farther to the center of rotation) this could lead to collapse of the building. The asymmetric buildings can be re-symmetrized by adding infill walls (which are coplanar with the frame) or closing openings.

- In steep sites, the space between poles should be filled with well made masonry walls in order to reduce the high asymmetry generated by the restraint exerted by the ground.

- Cantilevers. If possible, the cantilevered walls should be removed and rebuilt aligned with the nearest frame. In this way, the cantilever is replaced by balconies.
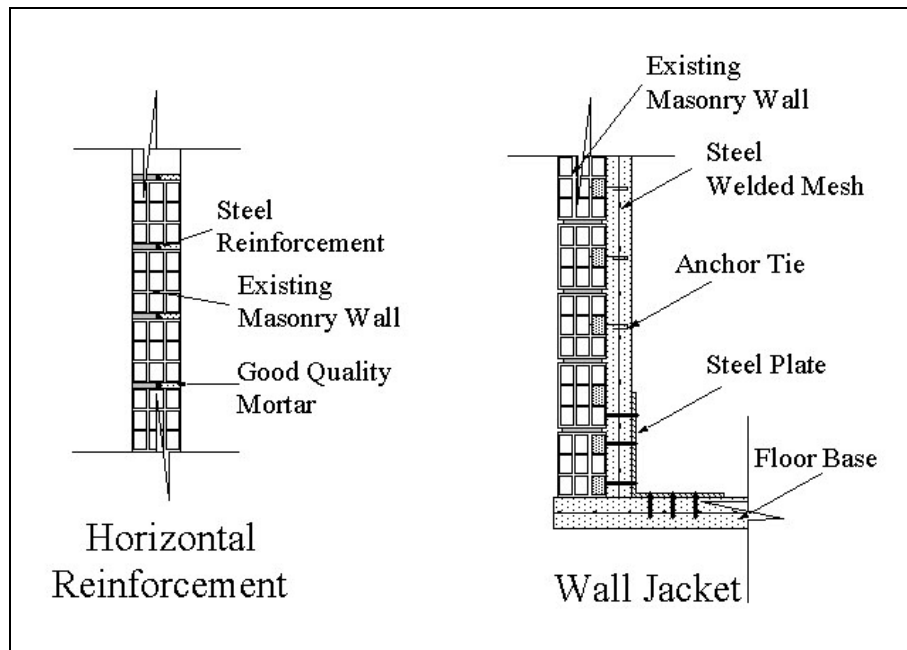

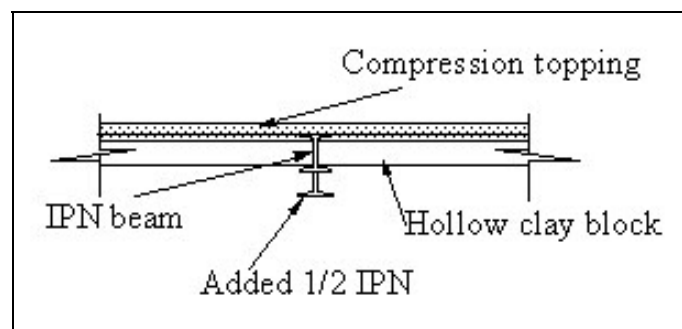**Figure 7.5: Longitudinal wall strengthening**

Figure 7.6: Secondary steel beams strengthening

Additional measures

- Do not erect additional floors. It applies even to single floor buildings.

- To avoid pounding between close buildings. If the slabs are aligned, some horizontal stiff elements (made of steel or timber) should be placed in between two adjoining slabs in order to avoid heavy pounding; to prevent falling of these elements during strong earthquakes they should be fixed to, at least, one of the buildings. Because of the difficulty of placing these elements, one of them could be installed from the front and other from the back side. Benefits: to avoid pounding and to join the seismic strengths of both buildings. If the slabs are unaligned (typically in steep sites), some vertical stiff elements (made of steel or timber) should be placed in between the two buildings in order

to avoid damaging pounding; these vertical elements will protect the walls and columns from stroke by the adjacent unaligned slab.

- 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. Benefit: to reduce the mass and, hence, the seismic forces.

- To fix the heavy and tall furniture and appliances to the floor or to the frame. Connections to the walls should be avoided because of the risk of out of plane failure.

- To avoid detachable roof parts. Heavy elements used to avoid sheathing uplift (bricks, rock, tires, among others), are neither enough nor advisable (because of the risk of fall and of the added mass).

- Excretal waters should be drained to the public sewage system to avoid possible soil problems due to local excess of water (especially in the lower parts of the premises).

7.2.2 Disregarded measures

This subsection contains some retrofit strategies which have been discarded because they are expensive, cumbersome, difficult to implement or have drawbacks or low reliability. In other contexts (mainly, in higher standards constructions) these solutions might be advisable.

- Frame strengthening through usual strategies: to line or jacket the members with composite materials (FRP), steel plates or reinforced concrete.

- Slab strengthening by adding an upper reinforced concrete layer. This measure has two relevant disadvantages: mass will be added and the lower slab components (steel beams) will be overloaded.

7.2.3 Seismic resistance of the strengthened buildings

The protection strategy to implement (the strengthening measures previously exposed) must provide an important reduction in the effects of these two earthquakes. For this purpose an empirical assessment of the proposed strengthening measures and its implications in vulnerability reduction are discussed in the following.

The strengthening conceptual framework is to provide additional resistance through the enhancement of wall resistance to shear stresses, as this measure is feasible to be performed in the context of the existing conditions. The strengthening assessment is performed in the building prototypes, adding the estimated additional shear strength that these walls supply.

The wall jacketing strategy renders excellent results in shear resistance, as the final strength may be many times (up to 3 times) greater than the resistance of the original wall. The recommended compressive strength of the concrete is $f_{ck} = 20$ MPa.

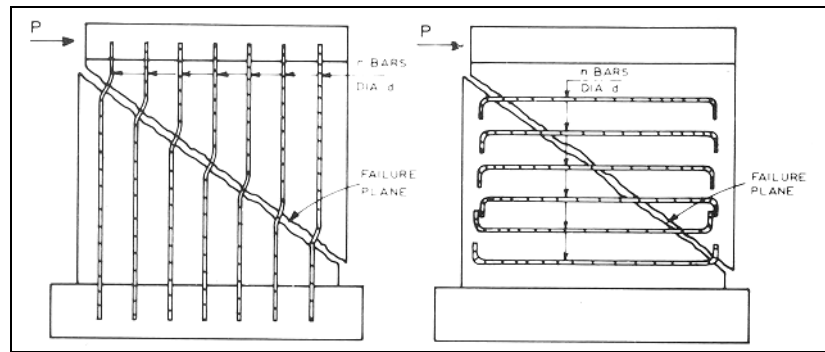The most critical failure mode is diagonal shear as shown by Figure 7.7.

Figure 7.7: Shear failure mode for the jacketed wall [Rosenblueth 1980]

Based in conservative assumptions, the shear strength of the jackets is calculated by means of the tensile capacity of the horizontal mesh rebars and the dowel capacity of the vertical ones, (not accounting for the capacity of the masonry or the concrete of the jacket), the expression to evaluate the shear resistance is:

$$V_j = C_{rh} \times A_{rh} \times f_{yk} / \gamma_s + C_{rv} \times A_{rv} \times f_{yk} / \gamma_s \qquad \text{eq. 7.1}$$

where V_j is the shear strength of the jacket, $C_{rh} = 0.9$ and $C_{rv} = 0.2$ are the reinforcement capacity reduction factors for horizontal and vertical reinforcements, respectively; A_{rh} and A_{rv} are the horizontal and vertical areas of the reinforcement, f_{yk} is the yielding point of the rebars and γ_s is a partial safety factor for steel (in this case it is assumed equal to 1) [City University of London, 2005].

The calculated shear resistance for each of the jacketing and directions (*x* and *y*) in the prototype buildings are shown in Table 7.1. The yielding point of the steel is $f_{yk} = 500$ MPa and the spacing between the bars is 0.1 m (in both directions).

Jacketing	Diameter of Welded Mesh bars (mm)	Shear Strength per wall *x* direction (kN)	Shear Strength per wall *y* direction (kN)
1	4	219.98	218.72
2	5	343	341.04
3	7	449.75	447.18

Table 7.1: Jacket strength for walls

The reinforced concrete coating yields two benefits: to provide the additional strength given by Table 7.1 and to confine the existing masonry wall thus allowing the formation of the full collapse mechanisms depicted at chapter 6. The lateral resistance of the wall-coating assembly is obtained adding those of the jackets and of the confined wall alone. This strength is enough to cope with the seismic demands. For instance, Table 7.2, Table 7.3 and Table 7.4, show the values of the lateral forces in buildings B1, B2 and B3, compared to those given by the strengthened walls.

Direction	Original Strength (kN)	Final Strength (jacket) (kN)	Demand (kN)
x (transversal)	273.84	$2 \times 219.98 + 273.84 = 713.80$	309.18
y (longitudinal)	336.00	$2 \times 218.72 + 336.00 = 773.44$	309.18

Table 7.2: Seismic resistance of strengthened building B1

Direction	Original Strength (kN)	Additional Strength (jacket) (kN)	Demand (kN)
x (transversal)	273.84	$2 \times 343 + 273.84 = 959.84$	562.18
y (longitudinal)	336.00	$2 \times 341.04 + 336.00 = 1018.08$	562.18

Table 7.3: Seismic resistance of strengthened building B2

Direction	Original Strength (kN)	Additional Strength (jacket) (kN)	Demand (kN)
x (transversal)	273.84	$4 \times 343 + 273.84 = 1645.84$	952.76
y (longitudinal)	336.00	$4 \times 341.04 + 336.00 = 1700.16$	952.76

Table 7.4: Seismic resistance of strengthened building B3

It is remarkable than the original strengths indicated in Table 7.2, Table 7.3 and Table 7.4 are bigger than those derived in chapter 6 since the jacketing allows the development of the whole wall strength.

7.2.4 Expected damage in the "La Milagrosa" Barrio after strengthening

After the retrofit of the buildings (according to the aforementioned recommendations), a new evaluation of the expected damage is performed following the LM1 methodology as in subsections 6.3.9.2 and 6.3.9.3. The buildings are re-classified as "Concrete Moment Frames with a Medium Code Level of Earthquake Resistant Design". The weighted average vulnerability index is $V_I^* = 0.469$.

Figure 7.8 shows the damage scenarios for input intensities VIII, and IX, respectively.
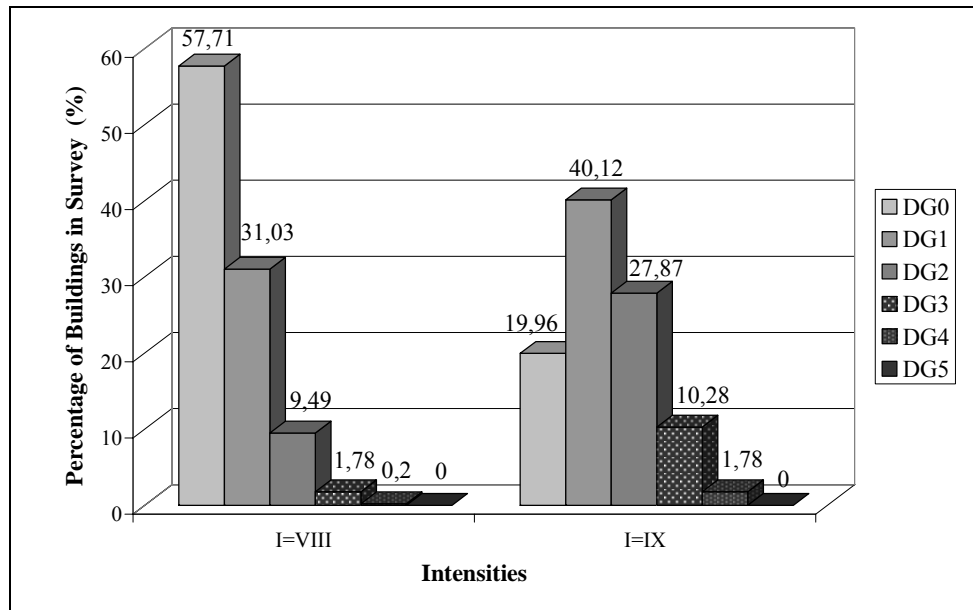
Figure 7.8: Damage grade distributions for I = VIII and I = IX

The percentages in Figure 7.8, show an important decrease in damage distribution, no collapsed buildings are expected for input events VIII and IX. Damage Grade 4 is reduced to less than a 1 % for I = VIII and less than a 2 % for I = IX.

7.3   Economical Appraisal

A required level of mitigation is the primary objective of strengthening, the choice of this particular level and the costs involved in its consecution are difficult to estimate, representing no straightforward decision-making process. Usually, earthquake defense measures are costly to implement and frequently involve public funds; it is also expected that, the higher the level of protection, the higher the cost. However, higher levels of protection will render in the future lower levels of losses (physical damage and lives), which can be accounted as a benefit to society. This last fact aims towards considering strengthening measures as investments which result in benefits to the society, and can be assessed in terms of cost-benefit analyses.

7.3.1 Cost-Benefit analysis

The cost-benefit analyses (CBA) are the most widely used approaches for choosing between alternative investments in strengthening projects, where they can help to make the choice of the buildings to be protected and the level of protection they should have. In simple words, CBA consists in estimating (in financial terms) the benefits of the project (reduction of damage and live losses), and deducting the costs of retrofitting, this difference is called the project's value [Coburn and Spence, 2002]. Any project rendering a positive value is worthwhile, but when a number of alternative projects are available and the financial resources are limited, the choice is based in the highest valued project.

To estimate the benefits, the distribution of expected damage and live losses must be assessed for both cases (actual state and strengthened buildings) in order to compare and obtain the

benefits of the project. Damage scenarios are then essential tools for estimating the future probable losses; moreover, for the actual state of the art, no other instruments are available.

7.3.2 Expected physical damage and losses

7.3.2.1 Present situation

For the "La Milagrosa" settlement, a damage and casualties scenario is assessed, through the LM1 methodology [Multinovic and Trendafiloski 2003], for the upper events (EMS intensities $I = $ VIII and $I = $ IX) and for the worst expected level of occupancy (occurring at midnight time) considered in this research. The results render a total loss of around 2 collapsed buildings and 31 buildings with unreparable damage for $I = $ VIII, and for $I = $ IX, 26 collapsed buildings and around 129 not reparable. The term unreparable implies that the building is no longer serviceable, as it has suffered very heavy damage (partial collapse in the upper floors), and most probably will have to be unoccupied as it represents a life threat for its inhabitants.

The economical losses are estimated as the present cost of reposition. It is composed of the debris removal, the construction cost (without including the bulk of the labor part) and a percentage of the value of the stored goods (mainly furniture and appliances). A gross average estimation of the total unit cost is 145 US$/m^2 (construction) + 5 US$/m^2 (debris) + 10 US$/m^2 (contents) = 160 US$/m^2. The price per building and per level is 14,400 US$, as the average construction area is around 90 m^2.

The percentages of buildings undergoing damage grades 4 and 5 are determined from the LM1 methodology [Multinovic and Trendafiloski 2003] and shown in Figure 6.31. Based in these figures (the number of damaged buildings and the cost of reposition) the physical damage can be estimated in financial terms as shown in Table 7.5 for both events. Collapsed buildings are those undergoing damage grade 5 and unreparable buildings have damage grade 4.

Intensity	Collapsed Levels (damage grade 5)	Unreparable Levels (damage grade 4)	Total Building Levels Lost	Economical Loss Estimate (US$)
$I = $ VIII	6	87	93	1,339,200
$I = $ IX	73	299	372	5,356,800

Table 7.5: Estimated cost of physical losses. Present situation

The figures in Table 7.5 for damage grade 4 correspond either to partial collapse or simply to unreparable damage without any significant collapse. This difference is relevant as partial collapse can generate casualties. It is estimated that only the floors (with damage grade 4) which are the upper ones in their buildings (i.e. they have no above floors) can experience partial collapse because, if a lower floor is partially collapsed, usually it leads to total collapse of the building. Assuming that the floors are uniformly damaged (i.e. the percentage of upper floors with respect to those unreparably damaged -but not collapsed- is the same as in the whole population –all the buildings in the Barrio "La Milagrosa"-), it is concluded that for $I = $

VIII, 32 floors (out of 87) are upper ones. For I = IX, 110 floors (out of 299) are upper ones. It is estimated that half of these upper floors undergo partial collapse: 16 floors for I = VIII and 55 floors for I = IX.

The number of victims, estimated based in a casualty model (explained in Section 8.8), are shown in Table 7.6 and Table 7.7 for damage grades 5 and 4, respectively.

Intensity	Injury Severity (people) for damage grade 5			
	Dead	Life Threat	Hospitalized	Light Injury
I = VIII	12	12	12	12
I = IX	161	150	161	150

Table 7.6: Estimated casualties for damage grade 5

Intensity	Injury Severity (people) for damage grade 4			
	Dead	Life Threat	Hospitalized	Light Injury
I = VIII	33	31	33	31
I = IX	114	106	114	106

Table 7.7: Estimated casualties for damage grade 4

The cost of saving a live is a difficult financial estimate, as life is considered priceless. Financial quantification is not performed here, although some countries have estimates of life saving; for example, in Turkey a cost-effectiveness study, performed by [Spence et al., 2002], rendered a cost per life ranging between US$ 250,000 and US$ 750,000.

7.3.2.2   *Strengthened buildings*

After the fulfillment of the strengthening operations (Section 7.2), the number of damaged buildings (undergoing damage grades 4 and 5) and of victims are re-estimated herein from the same models used in subsection 6.3.9.3. The numbers of seriously damaged levels and the subsequent economical losses are shown in Table 7.8.

Intensity	Collapsed Levels (damage grade 5)	Unreparable Levels (damage grade 4)	Total Building Levels Lost	Economical Loss Estimate (US$)
I = VIII	0	3	3	43,200
I = IX	0	27	27	388,800

Table 7.8: Estimated cost of physical losses. Strengthened buildings

Results from Table 7.8 show that the obtained results after retrofitting render no collapse of any building.

Considering the same assumptions than in Table 7.5 the number of partially collapsed levels (among those undergoing damage grade 4) are 1 for I = VIII and 5 for I = IX.

As the casualty model is based in the number of collapsed buildings, there are no victims (from dead to light injury) for damage grade 5. The number of victims, for damage grade 4 are shown in Table 7.9.

Intensity	Injury Severity (people) for damage grade 4			
	Dead	Life Threat	Hospitalized	Light Injury
I = VIII	2	2	2	2
I = IX	10	10	10	10

Table 7.9: Estimated casualties. Strengthened buildings

7.3.2.3 *Expected savings*

The differences between the figures in Table 7.5, Table 7.8 and Table 7.6, Table 7.7 and Table 7.9 are shown in Table 7.10 and Table 7.11 to highlight the lessening of victims and of damaged houses thanks to the strengthening operations.

Intensity	Collapsed Levels (damage grade 5)	Unreparable Levels (damage grade 4)	Total Building Levels Saved	Economical Savings Estimates (US$)
$I = VIII$	6 – 0 = 6	87 – 3 = 84	93 – 3 = 90	1,339,200 – 43,200 = 1,296,000
$I = IX$	73 – 0 = 73	299 – 27 = 272	372 – 27 = 345	5,356,800 – 338,800 = 5,018,000

Table 7.10: Estimated savings of cost of physical losses

Intensity	Injury Severity (people) for damage grades 4 and 5			
	Dead	Life Threat	Hospitalized	Light Injury
$I = VIII$	12 + 33 – 2 = 43	12 + 31 – 2 = 41	12 + 33 – 2 = 43	12 + 31 – 2 = 41
$I = IX$	161 + 114 – 10 = 265	150 + 106 – 10 = 246	161 + 114 – 10 = 265	150 + 106 – 10 = 246

Table 7.11: Estimated savings of casualties

7.3.3 Cost of the strengthening measures

The actual cost of the strengthening measures described in subsection 7.2.1 is grossly estimated herein. The assumed costs correspond to the present situation of the Venezuelan construction market. It has been assumed that the longitudinal walls are strengthened by one side jacketing. The labor cost is not included as these persons will be working for their own benefit.

The total cost is determined by estimating the retrofit costs of the prototype buildings B1, B2 and B3 (averaging B3-b and B3-c) and multiplying each of them by the numbers of buildings with one, two and three stories, respectively.

Table 7.12, Table 7.13, and Table 7.14 show the strengthening costs of prototype buildings B1, B2 and B3, respectively.

Walls (x) foundations	176.6
Walls (x) jacketing	268.28
Walls (y) foundations	264.9
Walls (y) jacketing	237.89
Walls topping (50%)	174.42
Walls moving (50%)	104.19
Roof fixing (50%)	163.8
Total Cost	**1,390.08**

Table 7.12: Retrofit cost of building B1 (US$)

Walls (x) foundations	176.6
Walls (x) jacketing	268.28
Walls (y) foundations	317.88
Walls (y) jacketing	237.89
Walls topping	348.84
Steel beam reinforcing	312.00
Walls moving (50%)	208.39
Cantilevers suppression (50%)	82.27
Roof fixing	202.50
Total Cost	**2,154.65**

Table 7.13: Retrofit cost of building B2 (US$)

Walls (x) foundations	211.92
Walls (x) jacketing	536.57
Walls (y) foundations	264.90
Walls (y) jacketing	475.78
Walls topping	348.84
Steel beam reinforcing	624.00
Walls moving (50%)	208.39
Cantilevers suppression (50%)	164.54
Roof fixing	202.50
Total Cost	**3,037.44**

Table 7.14: Retrofit cost of building B3 (US$)

The total cost of the seismic strengthening of the "La Milagrosa" Barrio is:

$$1,390.08 \times 162 + 2,154.65 \times 263 + 3,037.44 \times 81 = \mathbf{1,037,900 \ US\$}$$

7.3.4 Cost-Benefit results

An economical assessment of the benefits-investment ratio is presented herein after the results obtained in the previous subsections.

Table 7.15 presents the economical losses due to inputs of intensity VIII and IX, the same amounts assuming that the earthquakes come after the strengthening and the cost of such operation.

Intensity	Economical Loss (US$)	Economical Loss if strengthened (US$)	Prevention Cost (US$)
VIII	1,339,200	43,200	1,037,900
IX	5,356,800	338,800	1,037,900

Table 7.15: Expected economical losses after strengthening

Figures from Table 7.15 show that the ratio investment/benefit is positive even for the lowest intensity input and not accounting for the saved lives.

Recent information from the Venezuelan government [MCI, 2005] state that the construction cost of self-built dwellings (but possessing a higher level of quality than those in "La Milagrosa") is about 220 US$/m^2. As the total number of flats in "La Milagrosa" is 931, the cost of providing new houses to their inhabitants is

$$220 \times 931 = \textbf{18,620,000 US\$}$$

It is remarkable that this amount does not include the cost of the land. The comparison of this quantity with the investment required for seismic strengthening (1,037,900 US$) shows that it is significantly smaller.

Table 7.16 shows the number of saved dead and wounded.

Intensity	Saved lives	Saved injuries
VIII	45 – 2 = 43	131 – 6 = 125
IX	275 – 10 = 265	787 – 30 = 757

Table 7.16: Saved lives and injuries

This Table shows that a moderate investment can save an important number of lives.

7.4 Dissemination strategy

The measures drafted in this chapter are useless unless they are transmitted to their potential beneficiaries. Consequently, an intensive dissemination strategy (among all the involved parts) is strongly required. The concerned actors are community representatives, building officials, code makers, urban planners, constructors and the main construction products suppliers and manufacturers. Such strategy will consist of the following activities:

- Risk awareness. The involved parts must be extensively informed and aware of the actual level of risk. It is required a certain dread to guarantee enough receptivity.

- Technical information. The recommendation contained in this chapter will be upgraded, assessed and quantified to constitute reliable design and construction guidelines. The arising suggestions will be discussed with all the concerned actors to incorporate all the points of view. The final information will be issued mainly in two formats: a simple

illustrated brochure (mostly intended to near illiterate people) and a more comprehensive technical brochure (including the structural analysis justifying the proposed recommendations).

- Enforcement policy. The building officials will be urged to implement the application of these measures. Given the virtual impossibility of fulfilling the existing codes, new ones will be required (accounting for the special characteristics of the constructions). A suggested title is "Design guidelines for one and two levels self-constructed houses"; this norm should emphasize that the proposed measures are only temporary and palliative and do not guarantee the same level of safety than the normal codes. This will limit the legal responsibility of the code makers.