SASPARM 2.0

EUCENTRE European Centre for Training and Research in Earthquake Engineering

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Support Action for Strengthening PAlestine capabilities for seismic Risk Mitigation SASPARM 2.0

Deliverable D.C.1

Report on the identification of retrofit measures

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1 INTRODUCTION

1.1 General

Risk prevention envisages not only the compliance with Seismic Code for construction of new buildings, but strongly foresees the reduction of seismic risk through retrofitting of existing buildings in order to meet seismic safety requirements. Planning for enhancing the existing must be taken as the basis for the general urban development and building actions. New structures can be built to be sufficiently earthquake resistant by adopting proper design methodologies and construction quality control; whereas, on the contrary, the existing structures have been mostly planned without considering this important aspect. This trend poses enormous risk to human life resulting in losses and damages [1].

The need for seismic retrofitting in existing buildings can arise due to many reasons such as building not designed according to code, subsequent updating of code and design practice, deterioration of strength and aging, modification of existing structure or change in use.

Fundamental steps are prescribed in Eurocode 8 Part 3 [2] for the definition of one or more intervention strategies in a building and can be summarised as follows:

1. Mechanical and structural characteristics assessment: determination of the mechanical characteristics of materials, structural system and instability assessment and its causes;
2. Actions for security: isolation of area at risk, shoring, evaluation of structure practicability and its securing;
3. Planning and design choices evaluation.

Based on the assessment of the structure, its practicability and the extent of the damage, decision should be taken for the intervention. The technical criteria listed here below are thoroughly described in §5.1.2 of Eurocode 8, Part 3. Relevant aspects should be evaluated for the rehabilitation planning and are the following:

- Fixing all identified local gross errors appropriately remedied;
- Reducing the conditions that determine situations of strong irregularity of the buildings in terms of mass, strength and/or stiffness, also linked to the presence of non-structural elements;
- Improving the deformation capacity (local ductility) of individual elements;
- Increasing the shear resistance of the vertical elements;
- Reducing excessive deformation of horizontal elements;
- Improving links of non-structural elements;
- Improving foundation system (*e.g.* introduction of isolation system).

1.2 Scope

It is of critical importance that structures that need seismic retrofitting are identified correctly; attention has to be paid case by case, understanding the real deficiencies and failures. Moreover, considering that retrofitting of existing structures with insufficient seismic resistance accounts for a major portion of the total cost of hazard mitigation, and that it can be performed through several methods, an optimal intervention has to be conducted in a cost effective fashion.

In this report, potential rehabilitation techniques are provided, based on the taxonomy highlighted during field investigations, which identified four typologies. In particular, the taxonomy identification in an urban context is not only necessary for the vulnerability assessment, but also for retrofitting measures. Understanding the building peculiarities is helpful for planning retrofit campaigns, advising the best rehabilitation techniques and improving the seismic performance of existing buildings.

This report intends to assist engineers and stakeholders in reducing seismic risk and in adopting the retrofitting actions, with special regard to evaluation, costs and priorities. Indeed, construction cost is always important and is balanced against one or more considerations deemed significant. However, sometimes additional economic considerations, such as the cost of disruption to building users or the value of contents to be seismically protected, can be orders-of-magnitude larger than construction costs thus lessening its importance.

The present report covers retrofitting techniques with different levels of complexity; some of them may not be easily implemented at the moment but in the future their applicability is expected to be wider. The least applicable procedures in the actual Palestinian context are presented in Annex 1 and 2, which cover the fields of foundation systems, seismic isolation and dissipation. More details concerning the implementation of the proposed techniques can be found in Chapter 10.

A building-specific retrofitting study has as prerequisite an inelastic nonlinear static (pushover) or, even better, a dynamic (time-history) analysis. The acquisition of the necessary data to perform these analyses (plans, sectional detailing, material characteristics) can be a challenging procedure. The procedure requires experience, specific knowledge and software. The present report is not focused on how to conduct such analysis properly, since the region of application does not alter the principles of the above mentioned methodologies.

1.3 Overview

Chapter 2 gives a brief explanation of the building typologies identified during the field investigation. The definition of a building typology catalogue is the first step for large scale vulnerability assessment and retrofitting campaigns planning.

Chapter 3 describes typical seismic deficiencies and how they are placed into categories. A set of categories of seismic deficiencies is defined in order to better describe appropriate retrofit measures that are grouped in classes of rehabilitation techniques.

Chapter 4 illustrates general concepts for capacity-based design, mainly focusing on the explanation of “weak beam – strong column” hierarchy. Moreover, the short-column mechanism is described given that many buildings are affected by this weakness.

Chapters 5 to 8 identify the common seismic deficiencies for each building type. Mitigation techniques are suggested in the tables with reference to Chapter 9, in which rehabilitation techniques are described in detail.

Chapter 10 presents an overall evaluation of the presented retrofit techniques according to the availability of the used material, the familiarity, specific training requirements and efforts in structural analysis.

Finally, two annexes are presented: the first relates to foundations whereas the second refers to advanced techniques. The two mitigation methodologies are cross-cutting and for this reason, in addition to their innovative characteristic, can be applied to any building type.

2 MAIN BUILDING TYPOLOGIES

The building types that characterize the as built in Nablus are:

- Reinforced concrete frame buildings (C1);
- RC Frame buildings with soft storey (C1a);
- Shear wall buildings (C2);
- Masonry Buildings (URM).

Type C1 can be generically defined as a complete system of beams and columns supporting slabs (Figure 2.1). Regarding slab, two typologies have been identified: solid with drop beams that account for 3% of the in-built and ribbed slab with hidden beams, that is the most common (95%). Just a few portion of building (2%) has no beams at all. Lateral forces are resisted by cast-in-place moment frames that develop stiffness through rigid connections of the column and beams. Moreover, the presence of strong infills for the allocation of elevator and staircases increases the overall stiffness of the system.

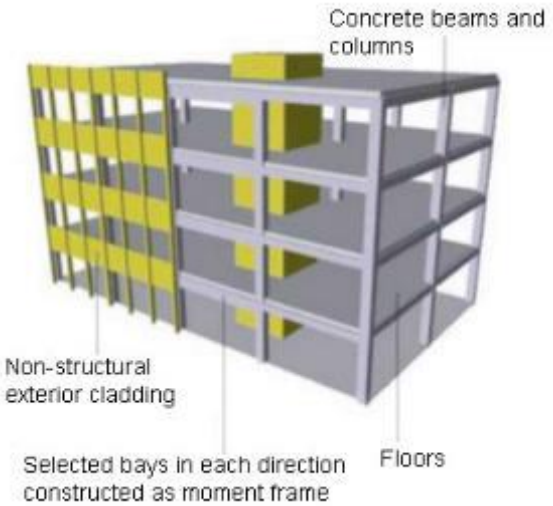


Figure 2.1. Reinforced concrete frame building (C1) [1].

RC buildings with soft storey (C1a) can be defined as a subclass of C1. They have the same structural characteristics of a reinforced concrete frame building, but with the peculiarity of lacking significant infill walls in a floor or in a side of it (Figure 2.2).

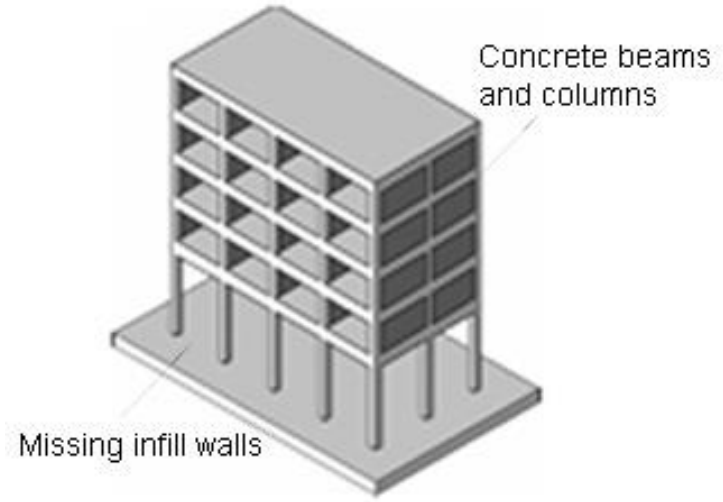


Figure 2.2. RC building with soft storey (C1a).

Building Type C2 is made of RC bearing walls (Figure 2.3); RC columns may be present providing strength for carrying gravity loads, but not participating much in lateral loads. Considering that C2 typology is new in the construction field of Nablus and it is usually applied for strategic building. It is the only one that fulfills requirements of Seismic Code, thus including earthquake loads for the design and boundary elements in the shear walls.

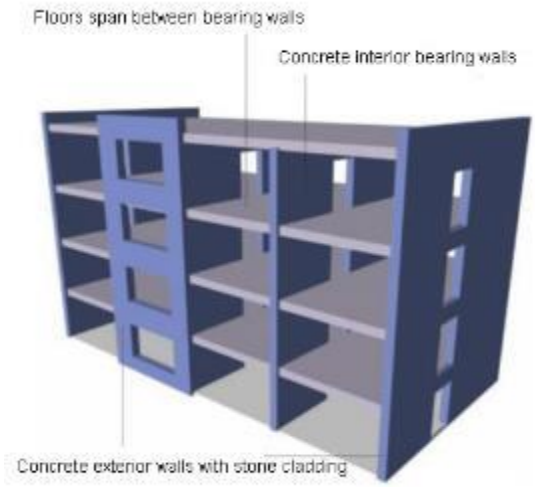


Figure 2.3. Shear wall building (C2) [1].

Finally, building Type URM (Figure 2.4) consists of unreinforced masonry bearing walls made up of stones in two layers with concrete in between or stones clades row-by-row with concrete casts behind them via formwork. In general, all walls act as both bearing and shear walls and the floors are concrete slabs cast-in-place.

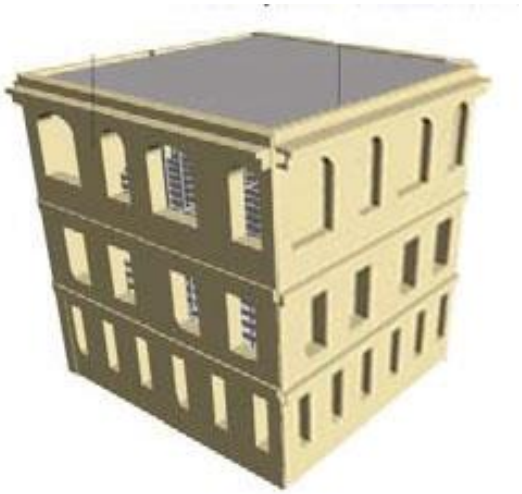


Figure 2.4. Unreinforced masonry building (URM) [1].

More details on the building types representative for Nablus can be found in DB1.

3 SEISMIC VULNERABILITY AND REHABILITATION

3.1 Category of seismic deficiencies

For developing strategies for seismic rehabilitation, the identified seismic deficiencies have been placed into *categories*. The categories of deficiencies that are present in an individual building will lead a user to consider certain techniques for rehabilitation.

3.1.1 Global Strength

Global strength typically refers to the lateral strength of the vertically oriented lateral force-resisting system at the effective global yield point.

A deficiency in global strength is common in older buildings either due to a complete lack of seismic design or a design to an early code with inadequate strength requirements and lack of proper plastic mechanism. Poorly detailed elements may become the critical component during the seismic event, as they generally possess inadequate resistance to reversed cyclic load. When these elements are subjected to cyclic inelastic deformation reversals, they undergo a fast deterioration and degradation of strength, failing in a brittle fashion with fatal consequences for the integrity of the structure as a whole.

3.1.2 Global Stiffness

Although strength and stiffness are often controlled by the same existing elements or the same retrofit techniques, the two deficiencies are typically considered separately.

Global stiffness refers to the stiffness of the entire lateral force-resisting system although the lack of stiffness may not be critical at all levels. For example, in buildings with narrow walls, critical drift levels occur in the upper floors. Conversely, critical drifts most often occur in the lowest levels in frame buildings. Stiffness must be added in such a way that drifts are efficiently reduced in the critical levels.

3.1.3 Configuration

This deficiency category covers configuration irregularities that adversely affect performance. In codes for new buildings, these configuration features are often divided into plan irregularities and vertical irregularities. Plan irregularities are features that may place extraordinary demands on elements due to torsional response or the shape of the diaphragm. Vertical irregularities are created by uneven vertical distribution of mass or stiffness between floors that may result in concentration of force or displacement at certain levels. In older existing buildings, such irregularities were seldom taken into consideration in the original design and therefore normally require retrofit measures to mitigate.

3.1.4 Sectional Detailing

Detailing, in this context, refers to design decisions that affect a component's or system's behaviour beyond the strength determined by nominal demand, often in the nonlinear range. In general, all potential early shear brittle failure of primary vertical elements should be eliminated. Perhaps the most common example of a detailing deficiency is poor confinement (stirrups strength or spacing) in concrete gravity columns. Often in older concrete buildings, the expected drifts from the design event will exceed the deformation capacity of such columns, potentially leading to degradation and collapse.

Another common example is a shear wall that has adequate length and thickness to resist the design shear and moment, but that has been reinforced such that its primary post-elastic behaviour will be degrading shear failure rather than more ductile flexural yielding. Identification of detailing deficiencies is significant in the selection of mitigation strategies because acceptable performance often may be achieved by local adjustment of detailing rather than by adding new lateral force-resisting elements.

3.1.5 Diaphragms

The primary purpose of diaphragms in the overall seismic system is to act as a horizontal beam spanning between lateral force-resisting elements. Deficiencies affecting this primary purpose, such as inadequate shear or bending strength, stiffness, or reinforcing around openings or re-entrant corners, are placed in this category.

3.1.6 Foundations

Foundation deficiencies can occur within the foundation element itself, or due to inadequate transfer mechanisms between foundation and soil. Element deficiencies include inadequate bending or shear strength of spread foundations; inadequate axial capacity or detailing of piles.
Transfer deficiencies include excessive settlement, excessive rotation, inadequate tension capacity of deep foundations, or loss of bearing capacity due to liquefaction.

3.2 Classes of Rehabilitation Measures

Once the deficiencies affecting the building are identified, retrofitting measures have to be taken. Rehabilitation techniques have been distinguished in four *classes*:

- Add elements, usually to increase strength or stiffness;
- Enhance performance of existing elements, increasing strength or deformation capacity;
- Reduce demand for those buildings with relatively weak lateral system and that also have excess space. The removal of several top floors can provide acceptable performance with economical and practical methods;
- Remove selected components, enhancing deformation capacity by uncoupling brittle elements or by removing them completely.

4 IMPORTANT DEVELOPMENTS IN RECENT SEISMIC DESIGN CODES

4.1 General Concepts of Capacity Based Design

Capacity design is a method of designing flexural capacities of critical member sections of a building structure based on behaviour of the structure in responding to seismic actions. This behaviour is reflected by the assumptions that the seismic action is of a static equivalent nature increasing gradually until the structure reaches its state of near collapse and critical regions occur simultaneously at predetermined locations to form a collapse mechanism simulating ductile behaviour. Ductility and energy dissipation of structure under depend mainly upon the vertical members (columns) of the structure. As far as the design is concerned, a key feature is to avoid undesirable modes of failure [3]. Capacity design procedure sets aside the results of analysis and aims at establishing a favourable hierarchy of strength in the structures, by ensuring that strength of columns is higher than that of adjacent beams ("weak beam strong column" concept), with possible allowance for beam overstrength. The area of greatest uncertainty of response of capacity design structures is the level of inelastic deformations that might occur under strong ground motions. It must be recognized that even with a "weak beam strong column" design philosophy, which seeks to dissipate seismic energy primarily in well-confined beam plastic hinges, a column plastic hinges must still form at the base of the column. In structure with "strong column weak beam" concept, beam yields first than column. As such, column sway mechanism is avoided in the structure.

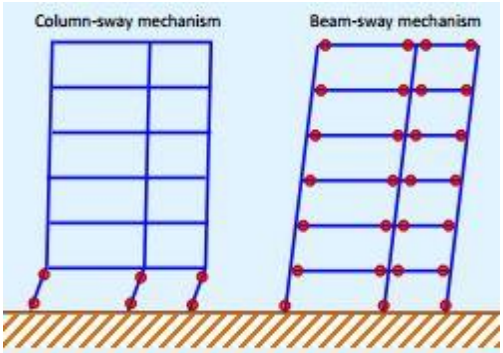


Figure 4.1. Possible collapse mechanisms for a frame: (left) column-sway collapse mechanism; (right) beam-sway collapse mechanism.

A comparison of the two example frames in Figure 4.1 shows that for the same maximum displacement at roof level, plastic hinges rotations in case (right) are much smaller than those in case (left). Therefore, the overall ductility demand, in terms of the large deflections, is much more readily achieved when plastic hinges develop in all the beams instead of only in the first storey column. The column hinge mechanism, shown in Figure 4.1 (left), also referred to as a soft-storey, may impose plastic hinge rotations, which even with good detailing affected regions, would be difficult to accommodate. This mechanism accounts for numerous collapses of framed buildings in recent earthquakes. A frame with "strong column weak beam" prohibits formation of column sway mechanism and only beam sway mechanism can be developed. As a result, a capacity design approach is likely to assure predictable and satisfactorily inelastic response under conditions for which even sophisticated dynamic analysis techniques can yield no more than crude estimates.

4.2 Short column mechanism

Many situations with short column effect arise in buildings. When a building is rested on sloped ground (Figure 4.2a), during earthquake shaking all columns move horizontally by the same amount along with the floor slab at a particular level (this is called rigid floor diaphragm action). The short column effect also occurs in columns that support mezzanine floors or loft slabs that are added in between two regular floors (Figure 4.2b).

There is another special situation in buildings when short-column effect occurs. Consider a wall (masonry or concrete) of partial height built to fit a window over the remaining height. The adjacent columns behave as short columns due to presence of these walls. In many cases, other columns in the same storey are of regular height, as there are no walls adjoining them. When the floor slab moves horizontally during an earthquake, the upper ends of these columns undergo the same displacement (Figure 4.3). However, the stiff walls restrict horizontal movement of the lower portion of a short column, and it deforms by the full amount over the short height adjacent to the window opening. On the other hand, regular columns deform over the full height [4].

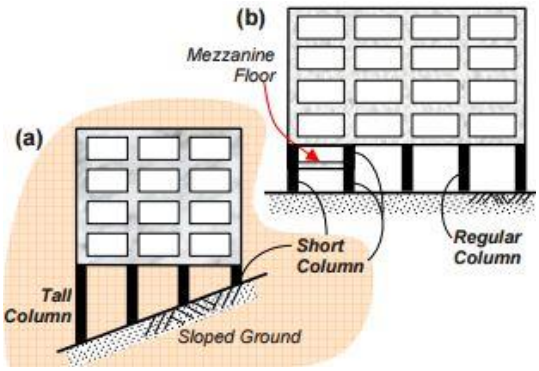


Figure 4.2. Buildings with short columns – two explicit examples of common occurrences.

Since the effective height over which a short column can freely bend is small, it offers more resistance to horizontal motion and thereby attracts a larger force as compared to the regular column. As a result, short column sustains more damage. Figure 4.4 shows X-cracking in a column adjacent to the walls of partial height.

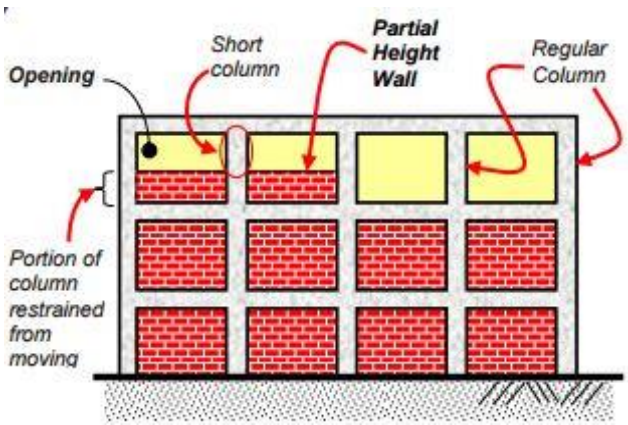


Figure 4.3. Short columns effect in RC buildings when partial height walls adjoin columns.

Figure 4.4. Short column effect due to partially filled frame opening.

Poor behaviour of short columns is due to the fact that in an earthquake, a tall column and a short column of same cross-section move horizontally by same amount Δ (Figure 4.5). However, the short column is stiffer as compared to the tall column, and it attracts larger earthquake force. Stiffness of a column means resistance to deformation – the larger is the stiffness, larger is the force required to deform it. If a short column is not adequately designed for such a large force, it can suffer significant damage during an earthquake. This behaviour is called Short Column Effect. The damage in these short columns is often in the form of X-shaped cracking – this type of damage of columns is due to shear failure. In new buildings, short column effect should be avoided to the extent possible during architectural design stage itself. When it is not possible to avoid short columns, this effect must be addressed in structural design. Several seismic codes for ductile detailing of RC structures require special confining reinforcement to be provided over the full height of columns that are likely to sustain short column effect.

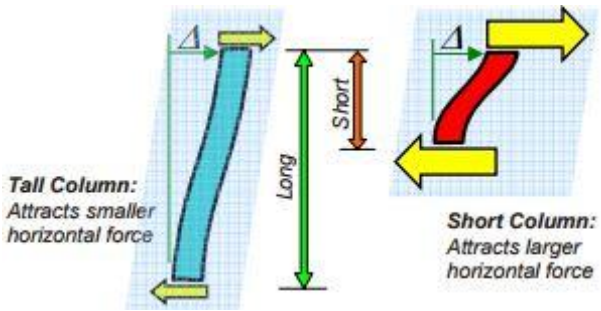


Figure 4.5. Cross-section horizontal seismic movement of columns.

5 BUILDING TYPE C1: REINFORCED CONCRETE MOMENT FRAME

These buildings consist of a complete system of RC beams and columns. Lateral forces are resisted by cast-in-place moment frames that develop stiffness through rigid connections of the column and beams. The lateral force-resisting frames could consist of the entire column and beam system in both directions. Floors may be a variety of cast-in-place reinforced concrete slabs (solid slab with drop beams or ribbed slabs with hidden slabs). External walls, generally interacting with the main structure, are made up of three layers (stone, concrete, blocks) or just hollow concrete blocks. They are not accounted for in the design for lateral loads, and the interaction with columns and beams could be beneficial or not.

Few shear walls could be present in the structure, for the formation of the elevator core and the support of the staircase slabs. These walls are not designed to resist to seismic forces. Their presence could often lead to torsional behaviour since they introduce strength/stiffness eccentricities (for more detail see “Torsional layout deficiency” below).

In older frame systems, usually not even designed for lateral loads, columns are frequently not stronger than beams or slabs, forcing initial yielding in the key vertical elements. On the other hand, the very few new RC frame buildings that are configured such that initial hinging occurs in the horizontal elements (capacity design principle), will exhibit stiffness and strength degradation and large drifts, but unless exceptionally weak, are less likely to collapse, since their plastic mechanism ensures larger energy dissipation.

Semi-ductile frames, with some but not all of current design features for concrete frames, will likely perform better, particularly if the columns are protected by shear failure and are designed to be flexurally controlled. However, even these more recent RC frames may not be able to reach medium ductility levels if a soft storey plastic mechanism is formed, due to architectural configuration or column layout (Cf. §6 for more details).
The brand new buildings of high importance with “fully ductile” frames and seismic detailing are expected to perform well, unless vertical or horizontal configuration irregularities concentrate inelastic deformation on certain structural components.

Table 5.1 presents a general summary of C1 main deficiencies grouped into categories as explained in Chapter 3.1. A detailed explanation of each deficiency is given afterwards. Therefore, several rehabilitation techniques are suggested and gathered into the four classes (Cf. §3.2), explained in detail in Chapter 9.

Figure 5.1. Examples of reinforced concrete frame buildings.

Table 5.1. Seismic deficiencies and potential rehabilitation techniques for C1 buildings					
Deficiency		Rehabilitation Technique			
Category	Deficiency	Add new elements	Enhance existing elements	Reduce demand (Advanced Techniques)	Remove selected components
Global strength/stiffness	Insufficient n° of frames	Steel X-braces [9.1] RC shear walls [9.2] Strong masonry infill walls [9.3]	Columns and/or beams: FRP jacketing* [9.4] RC jacketing [9.5] Steel jacketing [9.5]	Seismic Isolation* [Annex 2] Supplemental damping* [Annex 2]	
	Short – column mechanism	Masonry infill wall [9.3]	RC jacketing [9.5]		
	Infill walls failing or causing torsion	RC shear walls [9.2]	Uncouple infill walls		Remove infill walls
Configuration	Torsional layout (RC elevator core and staircases)	RC shear walls [9.2]			Remove RC shear walls
	Re-entrant corner	RC shear walls [9.2] New seismic joint* [9.18]			
Sectional detailing (ductility)	Weak column – strong beam		Columns and/or beams: FRP jacketing* [9.4] RC jacketing [9.5] Steel jacketing [9.5]		

Table 5.1. Seismic deficiencies and potential rehabilitation techniques for C1 buildings					
Deficiency		Rehabilitation Technique			
Category	Deficiency	Add new elements	Enhance existing elements	Reduce demand (Advanced Techniques)	Remove selected components
	Inadequate shear strength in column or beam		FRP jacketing* [9.4] RC/steel jacketing [9.5]		
	Insufficient flexural capacity (chord rotation)	Steel X braces [9.1] RC shear wall [9.2]	Column/Beams: FRP jacketing* [9.4] RC jacketing [9.5] Shear walls: FRP jacketing* [9.4]	Seismic Isolation* [Annex 2] Supplemental damping* [Annex 2]	
	Splices		FRP jacketing* [9.4] RC/steel jacketing [9.5]		
Diaphragms	Inadequate in-plane shear capacity		RC topping slab overlay [9.15] FRP overlays* [9.16]		
	Punching shear failure of slab-column connection		RC/steel jacketing [9.19]		
	Excessive stresses at openings and irregularities	Steel horizontal braces [9.17]	RC topping slab overlay [9.15] FRP overlays* [9.16]		Fill openings [9.14]
Foundation	Cf. Annex 1				

[Table 5.1 is adapted from [1]]

*More details concerning the applicability of the techniques proposed above can be found in §10 - Implementation in Palestine.

Each category of seismic deficiency with the related weakness are explained in the following sub-sections. The detailed illustration with the aid of figures and tables would lead to the accurate identification of the problem and to the consideration of certain techniques of rehabilitation, as recommended in Table 5.1. Therefore, efficient use of this report is dependent on the user understanding the nature of seismic deficiencies that may be characteristic of C1 typology.

Global Strength/ Stiffness

- Insufficient number of frames. Frames would be in one direction only and not related to each other in order to resist in both directions. Moreover, frame would present irregularities.

Figure 5.2. Frame irregularity example [6].

- Frames with inadequate stiffness: tall column with instability problem or short column with high concentration of shear stress (Cf. §4.2 for more details).

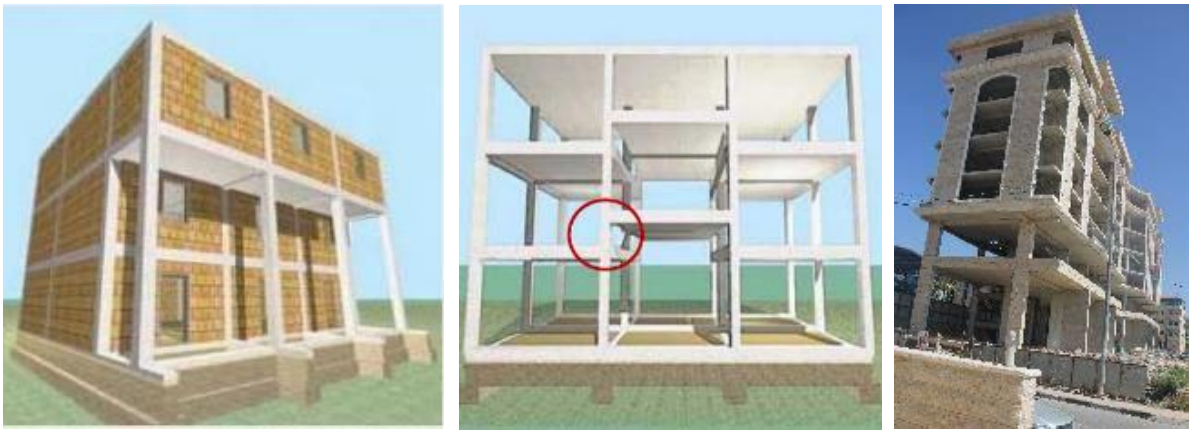


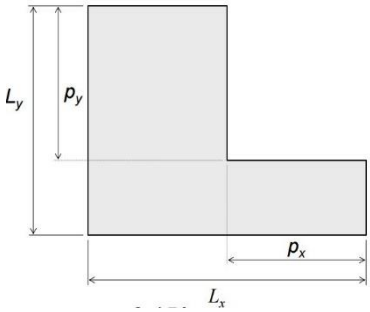
Figure 5.3. Tall column (left and centre) [6] and short column (right) examples.

- Infill walls failing or causing torsion.
  The walls can collapse out-of-plane because the fixing of the walls to the frame are not adequate, thus not collaborating and causing to the structure only mass addition.
  Infill walls on the RC frame structure with an inappropriate arrangement can cause asymmetrical behaviour. Due to the offset of the lateral rigidity centre of the floor (stiffness eccentricity in plan), the additional torsions cause the side and corner columns to be damaged easily under an earthquake.

Figure 5.4. Walls failing examples [6].

Configuration

- Re-entrant corner: eccentricity problems.
  Re-entrant corner arises in case of plans in H, I, T, L, C, U shapes. Presence of re-entrant corners are one of the serious plan irregularities that results in poor seismic performance of buildings. There are two major problems associated with re-entrant corners. The first is the torsion caused by the centre of mass and the centre of rigidity not coinciding, and the second is that they tend to produce differential motion between different wings of the building leading to local stress concentration at the re-entrant corner.



Irregularity for re-entrant corners exists if $p_x > 0.15\ L_x$ and $p_y > 0.15\ L_y$ [11]

Figure 5.5. Irregularity for re-entrant corner.

Figure 5.6. Examples of re-entrant corners [12] and Nablus building example (right-below).

- Torsional layout (RC elevator core and staircase).

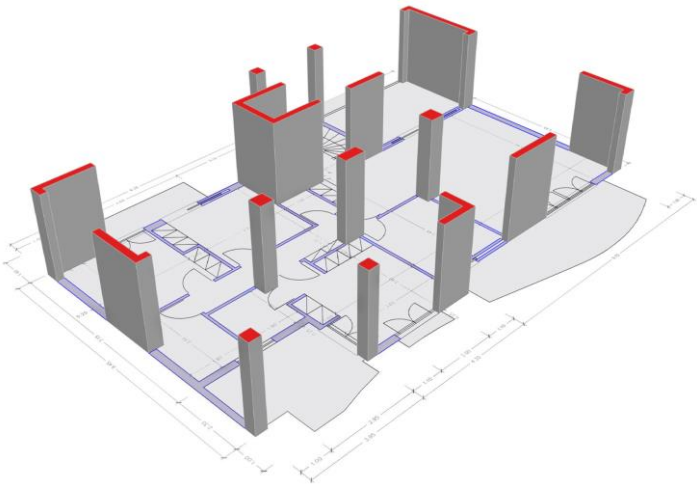


Figure 5.7. Torsional layout of RC shear wall elevator.

The centre of mass, or centre of gravity, of a building is the point at which it could be exactly balanced without any rotation resulting. If the mass (or weight) of a building is uniformly distributed (in plan), the result is that the plan's geometric centre will coincide with the centre of mass [5]. In a building, the main lateral force is contributed by the weight of the floors, walls, and roof and this force is exerted through the centre of mass, usually the geometric centre of the floor (in plan).

If the mass within a floor is uniformly distributed, then the resultant force of the horizontal acceleration of all its particles is exerted through the floor's geometric centre. If the resultant force of the resistance (provided by shear walls) pushes back through this point, dynamic balance is maintained.

Torsional forces are created in a building by a lack of balance between the location of the resisting elements and the arrangement of the building mass. This mechanism is called eccentricity between the centre of mass and the centre of resistance, which makes a building subjected to ground motion rotate around its centre of resistance, creating torsion in plan, which results in undesirable and possibly dangerous concentrations of stress.

In a building in which the mass is approximately evenly distributed in plan (uniform floor, wall and column masses), the ideal arrangement is that the earthquake resistant elements should be symmetrically placed, in all directions. With this distribution, the pushing direction is not relevant because the structure pushes back with a balanced stiffness that prevents rotation from trying to occur.

For this reason, it is recommended that buildings be designed as symmetrical as possible. In practice, some degree of torsion is always present, and the building code makes provision for this.

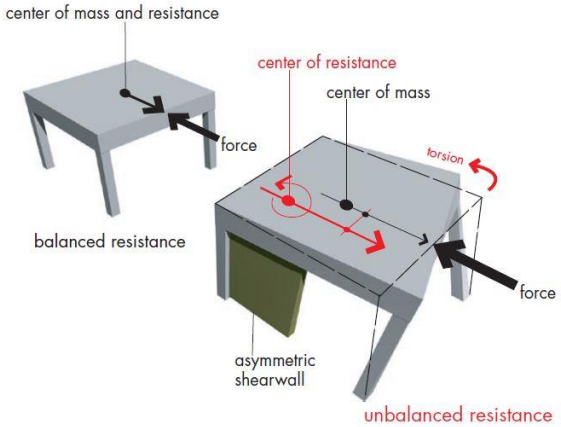


Figure 5.8. Balanced and unbalanced configurations [5].

Sectional Detailing (Ductility)

- Weak column – strong beam. Plastic hinge at the top or the base of the column due to an element with higher stiffness (Cf. § 4.1 for more details).

Figure 5.9. Weak column – strong beam examples [6].

- Inadequate transverse reinforcement in beams and/or columns that provides the necessary shear resistance, confinement of concrete and restraint against buckling of longitudinal reinforcement.

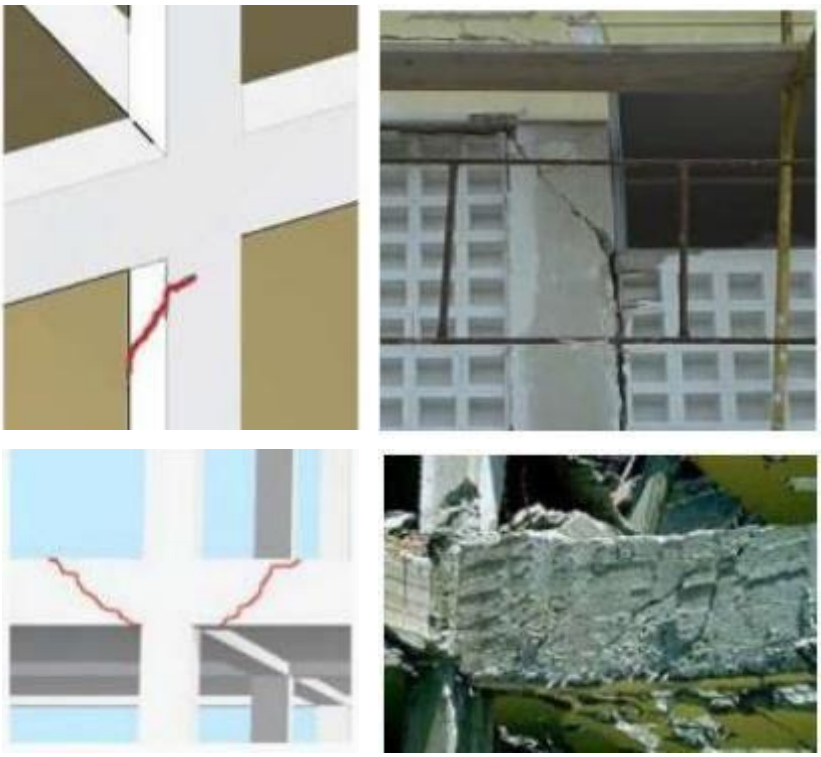


Figure 5.10. Inadequate shear strength examples [6].

- Insufficient flexural capacity. Inadequate confinement of beams/columns/walls in the critical regions where the plastic hinges form does not allow the sections to reach large curvature values and therefore limits the chord rotation capacity (ductility) and energy dissipation of the elements.

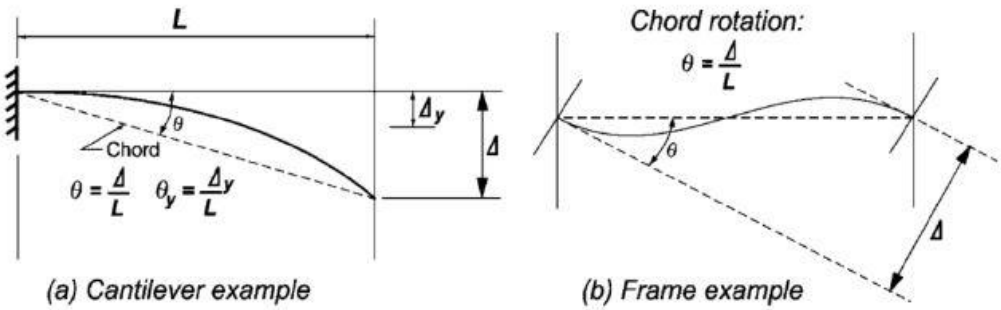


Figure 5.11. Definition of chord rotation (rotation angle).

- Splices. This failure mechanism is related to beam-column joint that has an inadequate reinforcement and/or an improper anchorage of longitudinal beam reinforcement. For this reason, the splice becomes a weak zone.

Figure 5.12. Splice crack example [6].

Diaphragms

- Punching shear failure of slab-column connection. Punching shear is a type of failure of reinforced concrete slabs subjected to high localized forces. In flat slab structures this occurs at column support points. The failure is due to shear. This type of failure is catastrophic because no visible signs are shown prior to failure.

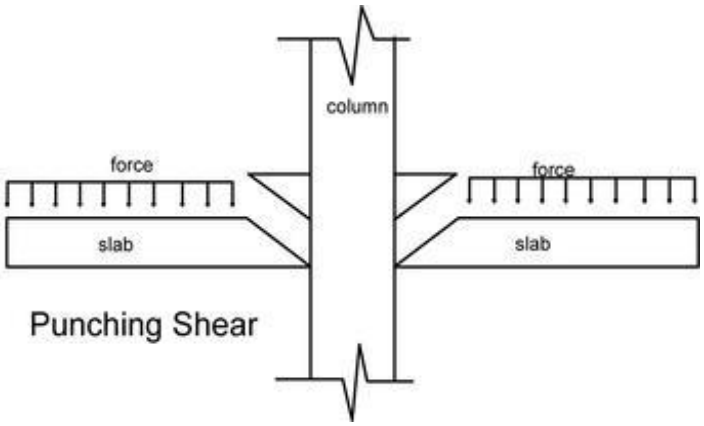


Figure 5.13. Punching shear failure of slab-column connection.

- Inadequate in-plane shear capacity. When the concrete stress reaches its tensile limit, diagonal cracks (at an angle of approximately 45°) occur across the slab.
  Another type of diaphragm failure is a direct shearing of the concrete along a line parallel to the deck corrugations. If the concrete covering is thin, this failure will be most likely to occur, with the ultimate shear strength depending on the shear strength of the concrete.

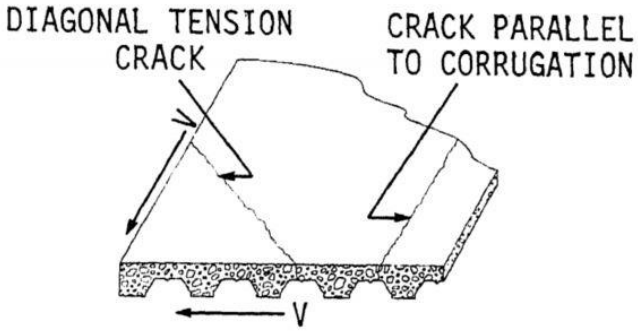


Figure 5.14. Crack examples in slab.

- Excessive stresses at openings and irregularities. For new structures, openings in the slab are usually determined in the early stages of design. On the other side, for existing structures analysis and strengthening methods are typically required for accounting new openings.
  Cutting openings in existing slabs should be approached with caution and avoided if possible. When cutting an opening in an existing slab, the effect on the structural integrity of the slab must be analysed. It is advisable to analyse the slab for excess capacity and possible moment redistribution before making the final decision on the sizes and locations of the openings [7].

Figure 5.15. Excessive opening example [7].

6 BUILDING TYPE C1A: REINFORCED CONCRETE FRAME BUILDINGS WITH SOFT STORY

This type of buildings comprises RC frame systems with an apparent soft story due to the absence of infill walls at least in one floor (not necessarily the ground one). The stiffness of the floor without infills is significantly smaller than the stiffness of the other floors. Moreover, irregular plan distribution of infill walls could also cause eccentricity issues that further compromise the structural performance.

The terms weak story and soft story are used when a story has less strength or stiffness than the story above or below. Soft story conditions occur when stiffness from one floor to the next changes abruptly. This is common at ground floors of commercial and office buildings with tall first stories with glass infills and no masonry walls. It could also occur at mid-heights of five-story to fifteen-story tall buildings that have not been designed for higher mode effects and near field motions. When a soft story is present, concentration of deformation demand occurs in it. Moreover, inadequate strength and story failure may occur. The ductility is much lower and so is the energy dissipation. The buildings fail at a much smaller drift. These buildings show the same deficiencies already analysed in Chapter 5 with addition of the soft storey deficiency for the configuration category.

Table 6.1 presents a general discussion of C1a main deficiencies grouped into categories, as explained in Chapter 3.1. Detailed explanations of each deficiency are provided next. Several rehabilitation techniques are suggested and gathered into the four classes (Cf. §3.2) - refer to Chapter 9 for the detailed measure explanation.

Figure 6.1. Building with soft story examples.

Table 6.1. Seismic deficiencies and potential rehabilitation techniques for C1a buildings					
Deficiency		Rehabilitation Technique			
Category	Deficiency	Add new elements	Enhance existing elements	Reduce demand (Advanced Techniques)	Remove selected components
Global strength/stiffness	Insufficient n° of frames	Steel X-braces [9.1] RC shear walls [9.2] Strong masonry infill walls [9.3]	Columns and/or beams: FRP jacketing* [9.4] RC jacketing [9.5] Steel jacketing [9.5]	Seismic Isolation* [Annex 2] Supplemental damping* [Annex 2]	
	Short – column mechanism	Masonry infill wall [9.3]	RC jacketing [9.5]		
	Infill walls failing or causing torsion	RC shear walls [9.2]	Uncouple infill walls		Remove infill walls
Configuration	Soft story mechanism	Steel X-braces [9.1] RC shear walls [9.2] Strong masonry infill walls [9.3]			
	Re-entrant corner	RC shear walls [9.2] New seismic joint* [9.18]			
	Torsional layout (RC elevator core and staircases)	RC shear walls [9.2]			Remove RC shear walls
Sectional Detailing (ductility)	Weak column – strong beam		Columns and/or beams: RC jacketing [9.5] Steel jacketing [9.5] FRP jacketing* [9.4		

Table 6.1. Seismic deficiencies and potential rehabilitation techniques for C1a buildings					
Deficiency		Rehabilitation Technique			
Category	Deficiency	Add new elements	Enhance existing elements	Reduce demand (Advanced Techniques)	Remove selected components
	Inadequate shear strength in column or beam		FRP jacketing* [9.4] RC/steel jacketing [9.5]		
	Insufficient flexural capacity (chord rotation)	Steel X braces [9.1] RC shear wall [9.2]	Column/Beams: FRP jacketing* [9.4] RC jacketing [9.5] Shear walls: FRP jacketing* [9.4]	Seismic Isolation* [Annex 2] Supplemental damping* [Annex 2]	
	Splices		FRP jacketing* [9.4] RC/steel jacketing [9.5]		
Diaphragms	Inadequate in-plane shear capacity		RC topping slab overlay [9.15] FRP overlays* [9.16]		
	Punching shear failure of slab-column connection		RC/steel jacketing [9.19]		
	Excessive stresses at openings and irregularities	Steel horizontal braces [9.17]	RC topping slab overlay [9.15] FRP overlays* [9.16]		Fill openings [9.14]
Foundation	Cf. Annex 1				

[Table 6.1 is adapted from [1]]

*More details concerning the applicability of the techniques proposed above can be found in §10 - Implementation in Palestine

Each category of seismic deficiency with the related weakness are explained in the following sub-sections. The detailed illustration with the aid of figures and tables would lead to the accurate identification of the problem and to the consideration of certain techniques of rehabilitation, as recommended in Table 6.1. Therefore, efficient use of this report is dependent on the user understanding the nature of seismic deficiencies that may be characteristic of C1a typology.

Configuration

- Soft story: different resistance between stories.
  Symmetry is not only required in plan of a building, but also, in structural configuration along the vertical direction. Abrupt variation in "storey" stiffness along the vertical direction shall be avoided. It generally arises when functional variations are planned. A typical example of this mechanism is an open/hollow ground floor for parking purposes; in such, ground storey possesses lower storey-stiffness to that of the upper storeys thereby triggering the so-called soft storey mechanism failure type. Development of plastic hinges at column ends accompanied by excessive storey drift in such soft storey are its typical failure mechanism.
  Building with shopping stores with large glass window openings at the ground floors also fall into this category. Besides, discontinuity in infill walls and variation in their location along the elevation also provides the ground for the development of soft storey. In such cases, an intermediate storey could also serve as soft storey if its stiffness is considerably lower than adjacent storeys.

Figure 6.2. Soft story examples [6].

7 BUILDING TYPE C2: SHEAR WALL SYSTEM

These buildings have a dual system of cast-in-place reinforced concrete walls that act as shear walls taking more than 70% of the seismic base shear force. This section covers the type of buildings where all RC walls usually act as both bearing and shear walls. The building may also include interior reinforced concrete columns that participate in carrying the gravity loads but their participation in carrying the lateral load is small.

As in C1 building type, shear walls are used for forming the elevator core and supporting staircases. Even if additional masses are present, torsional behaviour is less likely to happen due to a better balance in plan configuration (rigid walls members in the perimeter).

In order for this system to be efficient, a regular and repeating pattern of concrete walls is required to provide support points for the floor framing.

The parallel layouts of supporting walls normally lead to the use of one-way uniform-depth concrete floor systems. Cast-in-place systems have been generally employed and the adequacy of the shear connection between slab and walls is critical for the whole system.

When subjected to ever increasing lateral load, individual shear walls or piers will first often force yielding in spandrels, slabs, or other horizontal components restricting their drift, and eventually either rock on their foundations, suffer shear cracking and yielding, or form a flexural hinge usually near the base and less frequently at mid-height. Yielding of spandrels, slabs, or other coupling beams can cause a significant loss of stiffness in the structure. Flexural yielding will tend to maintain the strength of the system, but shear yielding will degrade the strength of the coupling component and the individual shear wall or pier will begin to act as a cantilever from its base. In this building type, the coupling elements are often slabs, and their lack of bending stiffness may reduce or eliminate significant coupling action.

Discontinuous walls in height must be avoided since they create load transfer deficiency. In such cases, collectors are often needed in the floor diaphragm, and supporting columns need axial strengthening.

Figure 7.1. Examples of shear walls buildings.

Due to the extensive use of walls, buildings of this type seldom have deficiencies in Global Strength and Stiffness categories, unless significant degradation of strength occurs due to shear failures.

Moreover, the effect of coupling beams on initial stiffness and the potential change in stiffness due to yielding of these coupling wall-beams over doors should be investigated (Cf. Sectional Detailing – Brittle failure for more details).

Table 7.1 provides a summary of the main deficiencies for the class C2, grouped into categories as explained in Chapter 3.1. Detailed explanations for each deficiency are given following the table. Therefore, several rehabilitation techniques are suggested and gathered in four classes (Cf. §3.2). Chapter 9 includes the detailed explanation for such measures.

Table 7.1. Seismic deficiencies and potential rehabilitation techniques for C2 buildings					
Deficiency		Rehabilitation Technique			
Category	Deficiency	Add new elements	Enhance existing elements	Reduce demand (Advanced Techniques)	Remove selected components
Sectional Detailing	Insufficient in-plane wall shear strength (web or boundary element)	Steel X braces [9.1] RC shear wall [9.2]	FRP jacketing* [9.4] RC jacketing [9.5]	Seismic Isolation* [Annex 2] Supplemental damping* [Annex 2]	
	Insufficient flexural capacity (chord rotation)	Steel X braces [9.1] RC shear wall [9.2]	Column/Beams: FRP jacketing* [9.4] RC jacketing [9.5] Shear walls: FRP jacketing* [9.4]	Seismic Isolation* [Annex 2] Supplemental damping* [Annex 2]	
	Brittle failure of coupling beams	RC shear wall [9.2]	Beams: FRP jacketing* [9.4] RC jacketing [9.5] Steel jacketing [9.5]	Seismic Isolation* [Annex 2] Supplemental damping* [Annex 2]	
Configuration	Torsional layout (RC elevator core and staircases)	Concrete shear walls [9.2]			Remove RC shear walls
	Re-entrant corner	Concrete shear walls [9.2] New seismic joint* [9.18]			

Table 7.1. Seismic deficiencies and potential rehabilitation techniques for C2 buildings					
Deficiency		Rehabilitation Technique			
Category	Deficiency	Add new elements	Enhance existing elements	Reduce demand (Advanced Techniques)	Remove selected components
	Discontinuous walls	Concrete shear wall [9.2] or adequate columns beneath	Concrete/steel jacket of supporting columns [9.5]		Remove wall
Diaphragms	Inadequate in-plane shear capacity		RC topping slab overlay [9.15] FRP overlays* [9.16]		
	Excessive stresses at openings and irregularities	Steel horizontal braces [9.17]	RC topping slab overlay [9.15] FRP overlays* [9.16]		Fill openings [9.14]
Foundation	Cf. Annex 1				

[Table 7.1 is adapted from [1]]

*More details concerning the applicability of the techniques proposed above can be found in §10 - Implementation in Palestine.

Each category of seismic deficiency with the related weakness are explained in the following sub-sections. The detailed illustration with the aid of figures and tables would lead to the accurate identification of the problem and to the consideration of certain techniques of rehabilitation, as recommended in Table 7.1. Therefore, efficient use of this report is dependent on the user understanding the nature of seismic deficiencies that may be characteristic of C2 typology.

Sectional Detailing

- Insufficient in-plane wall shear strength. Inadequate transverse reinforcement in web (rebars) and/or boundary elements (stirrups) to provide the necessary shear resistance. It leads to early brittle failure of crucial elements.

Figure 7.2. Shear failure of RC wall.

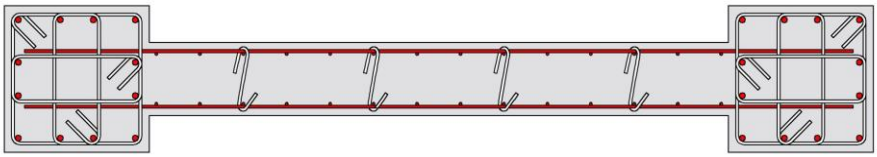


Figure 7.3. Modern sectional detailing of RC shear wall.

- Insufficient flexural capacity. Inadequate confinement of beams/columns/walls in the critical regions where the plastic hinges form does not allow the sections to reach large curvature values and therefore limits the chord rotation capacity (ductility) and energy dissipation of the elements.

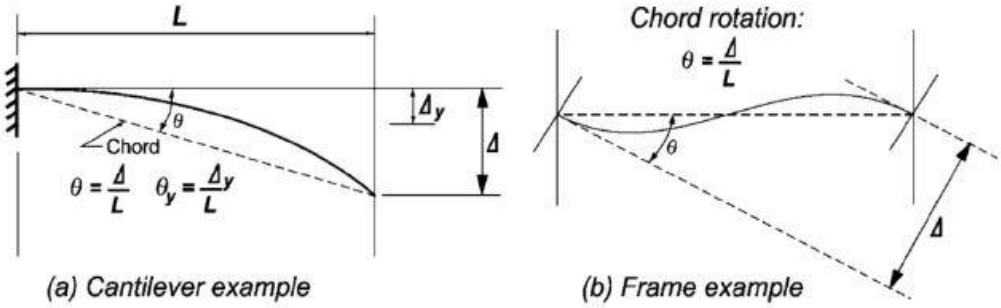


Figure 7.4. Definition of chord rotation (rotation angle).

- Brittle failure of coupling beams. Coupled shear walls consist of two shear walls connected intermittently by beams along the height. The behaviour of coupled shear walls is mainly governed by the coupling beams. The primary purpose of beams between coupled walls during earthquake actions is the transfer of shear from one wall to the other. In considering the behaviour of coupling beams it should be appreciated that during an earthquake significantly larger inelastic excursions can occur in such beams than in the walls that are coupled. During one earthquake, larger numbers of shear reversals can be expected in the beams than in the walls. Many coupling beams have been designed as conventional flexural members with stirrups and with some shear resistance allocated to the concrete. Such beams will inevitably fail in diagonal tension. It is evident that the principal diagonal failure crack will divide a relatively short beam into two triangular parts. Unless the shear force associated with flexural overstrength of the beam at the wall faces can be transmitted by vertical stirrups only, diagonal tension failure will result [8].

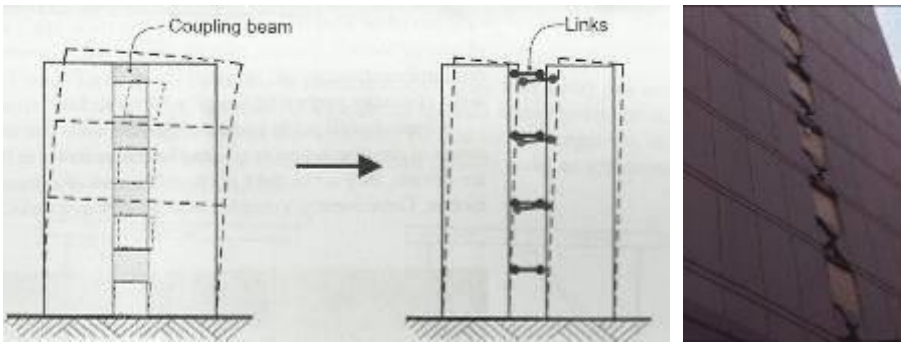


Figure 7.5. Shear failure of coupling beams.

Configuration

- Torsional layout – RC elevator core and staircases. Shear walls are concentrated around elevator shafts and staircases. This arrangement lacks good torsional response due to stiffness eccentricity.
  If the core remains in a position leading to torsional behavior as in Figure 7.6, then it must be designed explicitly for torsion (strong coupling beams). It is far preferable to adopt a symmetrical arrangement to avoid this. (Cf. §5 – Torsional layout for more details).

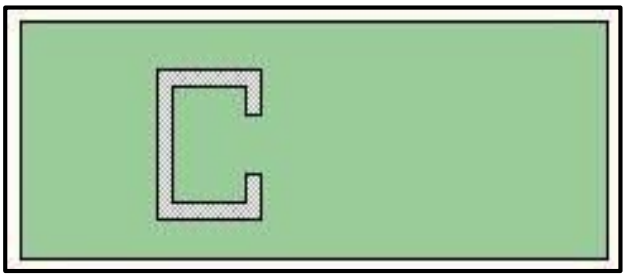


Figure 7.6. Torsional layout configuration.

- Re-entrant corner. Cf §5 – Configuration for more details.
- Discontinuous walls: serious overstressing at the points of discontinuity due to the non-continuity of the load path through the walls from the roof to foundation.

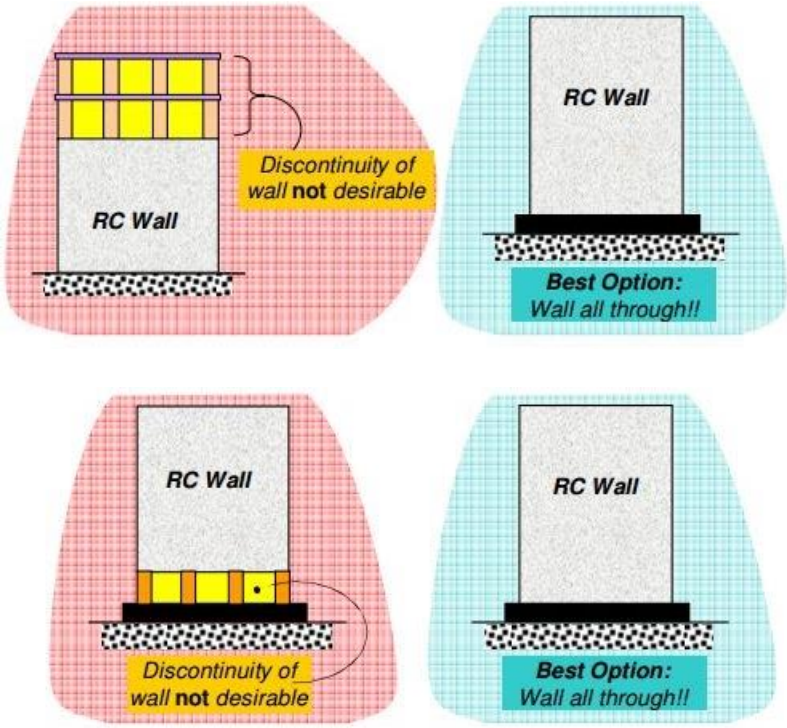


Figure 7.7. Examples of discontinuous walls.

Shear walls are designed to receive lateral forces from diaphragms and transmit them to the ground. The forces in these walls are predominantly shear forces in which the material fibers within the wall try to slide past one another. To be effective, shear walls must run from the top of the building to the foundation with no offsets and a minimum of openings [9].
If a discontinuity is present in the load path, the building is unable to resist seismic forces regardless of the strength of the existing elements. The design professional should be watchful for gaps in the load path (*e.g.* discontinuity in height).
In cases where there is a structural discontinuity, a load path may exist but it may be a very undesirable one. At a discontinuous shear walls, for example, the diaphragm may transfer the forces to frames not intended to be part of the lateral-force-resisting system.

Diaphragms

- Inadequate in-plane shear capacity. Cf §5 – Diaphragms for more details.
- Excessive stresses at openings and irregularities. Cf §5 – Diaphragms for more details.

8 BUILDING TYPE URM: UNREINFORCED MASONRY BEARING WALLS

Building Type URM consists of unreinforced masonry bearing walls, usually at the perimeter and usually brick masonry. The floors are typically of reinforced concrete: two-way solid slab or steel joist as supporting beams.

It has consistently performed poorly in earthquakes. The most common failure is an outward collapse of the exterior walls caused by loss of lateral support due to separation of the walls from the floor and roof diaphragms.

Stone unit masonry is the most common type of masonry unit, but there are a number of other common types, such as hollow clay brick, structural clay tile, concrete masonry. Stone masonry is usually made of limestone. The cavity in blocks reduces the out-of-plane seismic capacity of the wall.

Material properties—such as compressive, tensile and shear strengths and compressive, and tensile and shear moduli—vary widely among masonry units, brick and mortar. An important issue for in-plane capacity is the relative strength of masonry and mortar. When mortars are stronger than the masonry, strength may be enhanced, but brittle cracking through the masonry units may be more likely to occur, resulting in lower deformation capacity.

URM buildings are generally considered to be one of the most hazardous building types. Significant property damage and loss of life have occurred in URM buildings during earthquakes around the world. The primary deficiencies are due to unbraced parapets which can fall on adjacent pedestrian thoroughfares and poorly connected walls and diaphragms which can lead wall failure and loss of vertical support for diaphragms.

As shear wall buildings, global strength in URM buildings is dependent on the in-plane shear capacity of the walls. When walls are found to be deficient, new vertical lateral force-resisting elements can be added at interior locations or existing walls can be enhanced. At interior locations, new elements include wood structural panel shear walls, concrete shear walls, reinforced masonry shear walls, braced frames, and moment frames. At exterior locations, care must be taken to address relative rigidity concerns. Typically, concrete or shotcrete overlays are used to enhance the URM wall capacity.

URM bearing walls are generally quite rigid. When walls are solid or lightly punctured with window openings, global stiffness deficiencies are typically not an issue. In some buildings, though, facades facing the street can be highly punctured with relatively narrow piers between openings. In addition to lacking adequate strength, these wall lines may also be too flexible as well.

Many commercial URM buildings will have a fairly open street façade at the ground level, leading to a weak and soft first story and torsional irregularities. This is usually addressed by the addition of a moment frame at the façade or another vertical lateral force-resisting element at some distance back from the façade.

It is the lack of adequate ties between the walls and diaphragms that is the single most significant deficiency in URM buildings. Rehabilitation measures include tension ties for out-of-plane forces and shear ties for in-plane forces.

Since the masonry elements in Building Type URM are unreinforced by definition, they do not comply with modern ductile detailing requirements. Walls deemed susceptible to out-of-plane bending failures can be strengthened by strongbacks placed against them either on the outside or more commonly on the interior face.

Table 8.1 presents a general discussion of URM main deficiencies grouped into categories as explained in Chapter 3.1. Detailed explanations of each deficiency are given following the table.

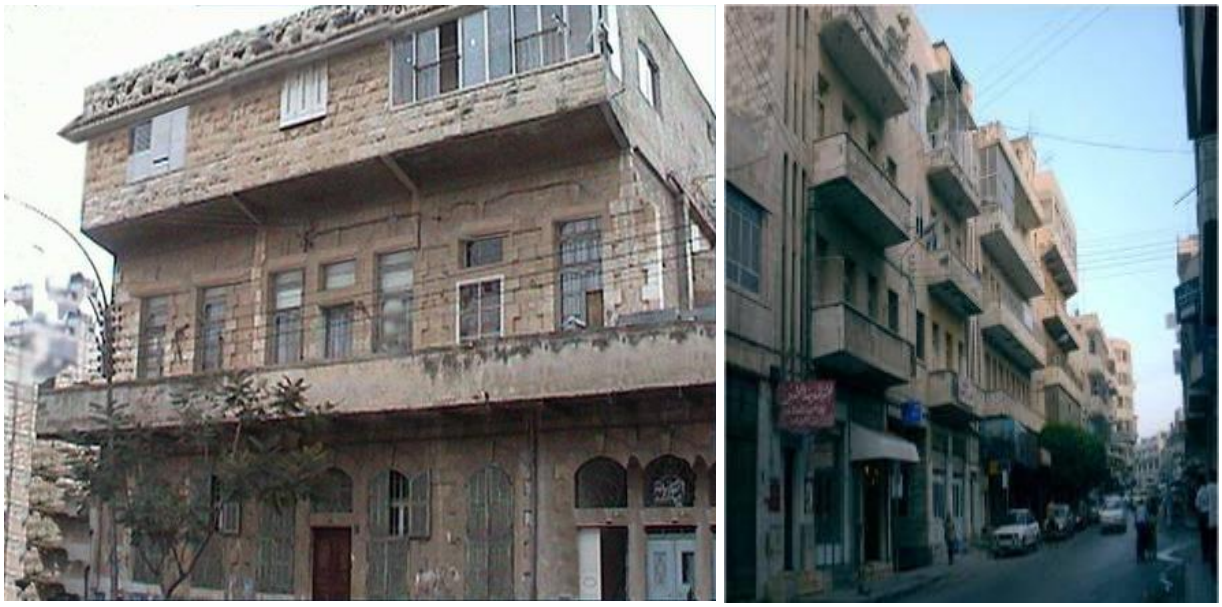


Figure 8.1. Masonry buildings examples.

Table 8.1. Seismic deficiencies and potential rehabilitation techniques for URM buildings					
Deficiency		Rehabilitation Technique			
Category	Deficiency	Add new elements	Enhance existing elements	Reduce demand (Advanced Techniques)	Remove selected components
Global Strength	Insufficient in-plane wall strength	Steel braced frame [9.1]	RC jacketing [9.9] FRP jacketing* [9.10] Grouting infill openings [9.11]	Seismic Isolation* [Annex 2]	
Configuration	Excessive torsion	Braced frame [9.1] Concrete/masonry shear wall [9.2; 9.3]			
Structural Detailing	Wall inadequate for out-of-plane bending	Out-of-plane bracing [9.7] Add diagonal/vertical bracings [9.13]	Reinforced cores [9.8] Concrete wall overlay [9.9] FRP overlay* [9.10]		
	Unbraced parapet		Brace parapet [9.6]		Remove parapet [9.6]
	Poorly anchored veneer or appendages		Add veneer ties [9.13]		Remove veneer or appendages
Diaphragms	Inadequate in-plane strength and stiffness	Steel horizontal braces [9.17]			
	Inadequate chord capacity	Steel horizontal braces [9.17]			

Table 8.1. Seismic deficiencies and potential rehabilitation techniques for URM buildings					
Deficiency		Rehabilitation Technique			
Category	Deficiency	Add new elements	Enhance existing elements	Reduce demand (Advanced Techniques)	Remove selected components
	Excessive stresses at openings and irregularities	Steel horizontal braces [9.17]			
Foundation	Cf. Annex 1				

[Table 8.1 is adapted from [1]]

*More details concerning the applicability of the techniques proposed above can be found in §10 - Implementation in Palestine.

Each category of seismic deficiency with the related weakness are explained in the following sub-sections. The detailed illustration with the aid of figures and tables would lead to the accurate identification of the problem and to the consideration of certain techniques of rehabilitation, as recommended in Table 8.1. Therefore, efficient use of this report is dependent on the user understanding the nature of seismic deficiencies that may be characteristic of URM typology.

Global Strength

- Insufficient in-plane wall strength. The capability of URM walls to resist lateral loads is limited by the strength of both masonry units and bed joint mortar. Such unreinforced walls are generally incapable of withstanding severe repeated load reversals, suffering from low energy dissipation capacity and severe strength degradation characteristics as the double-diagonal (X) shear cracking develops into extensive damage.

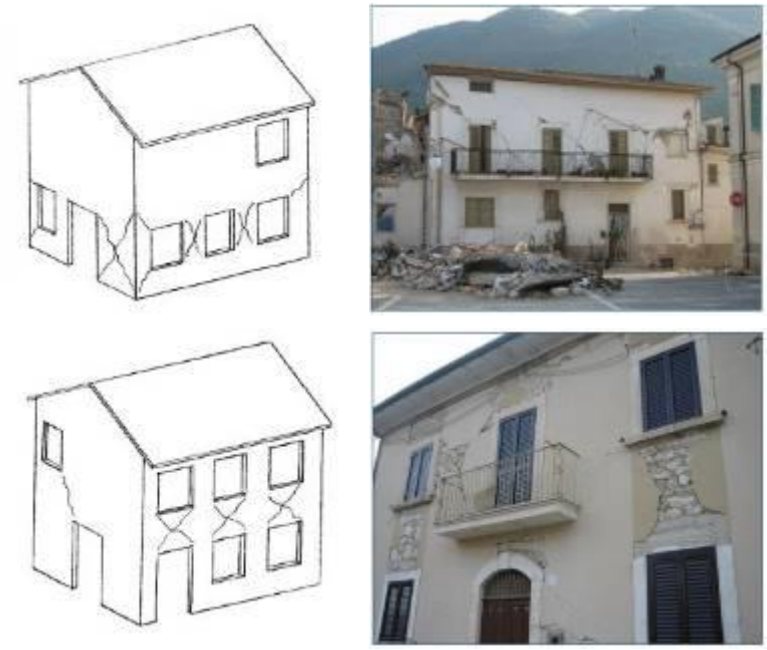


Figure 8.2. Examples of insufficient wall strength [6].

Configuration

- Excessive torsion. Symmetrical buildings of relatively simple form usually perform better than complex shapes where walls are asymmetrically distributed on the plan [10].

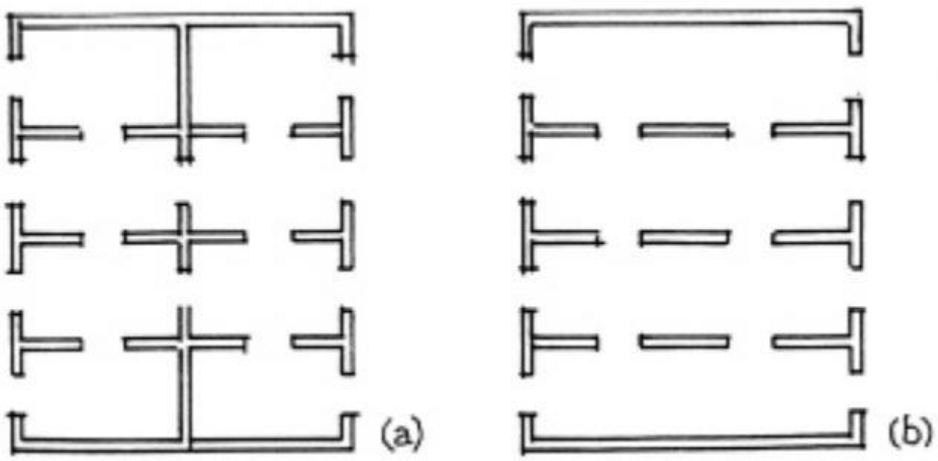


Figure 8.3. Distribution of walls in plan (a) adequate – (b) not adequate.

Structural Detailing

- Wall inadequate for out-of-plane bending. This kind of damage most often occurs due to the instability of structural elements. Once the walls have been anchored to the diaphragms, the unbraced length of these brittle walls may cause them to buckle out-of-plane during lateral loading. Additional bracing should be utilized between diaphragms to increase the out-of-plane strength of the URM wall assembly.

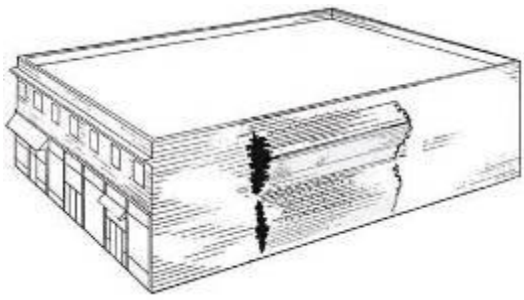


Figure 8.4. Out-of-plane failure [26].

- Unbraced parapet. A lot of damage to URM buildings can be attributed to parapet failures when a seismic event occurs. The damage can occur not only to the building itself, but to nearby buildings, and poses a major risk to life safety. Bracing the parapets is a must when evaluating the needed retrofitting of a URM building.

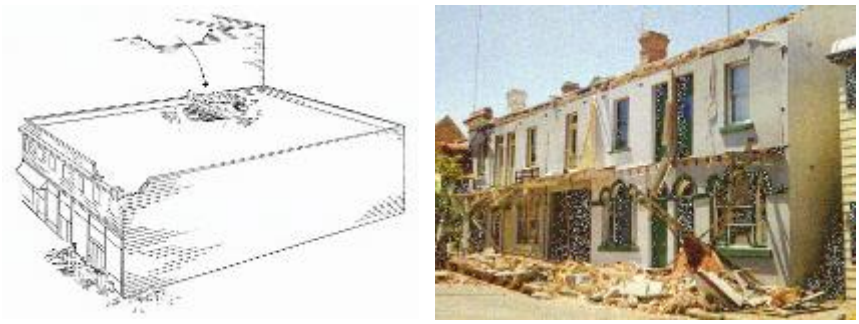


Figure 8.5. Parapet failing [26].

- Poorly anchored veneer or appendages. Non-structural elements can cause damages, it is important to perform better anchorage.

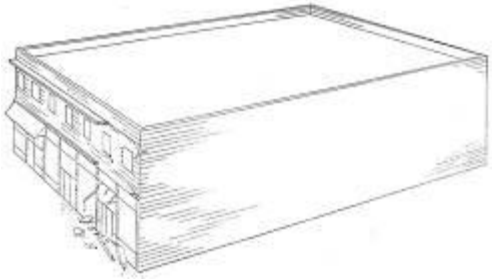


Figure 8.6. Veneer failing [26].

Diaphragms

- Inadequate in-plane strength and stiffness**.**

Figure 8.7. 2005 Pakistan earthquake.

- Inadequate chord capacity. The URM walls must be tied to the horizontal diaphragms (roof and floors) to increase their resiliency to out-of-plane loading and catastrophic failures.

Figure 8.8. Inadequate chord capacity examples [6].

- Excessive stresses at openings and irregularities. Cf. §5 – Diaphragms for more details.

9 REHABILITATION TECHNIQUES

9.1 Add Steel X-braces (Connected to a Concrete Diaphragm)

Addition of steel diagonal braces to an existing concrete moment frame building is a method of adding stiffness to the structural system. The steel braces can be added without a significant increase in the building weight. Figure 9.1 shows several common configurations [13].

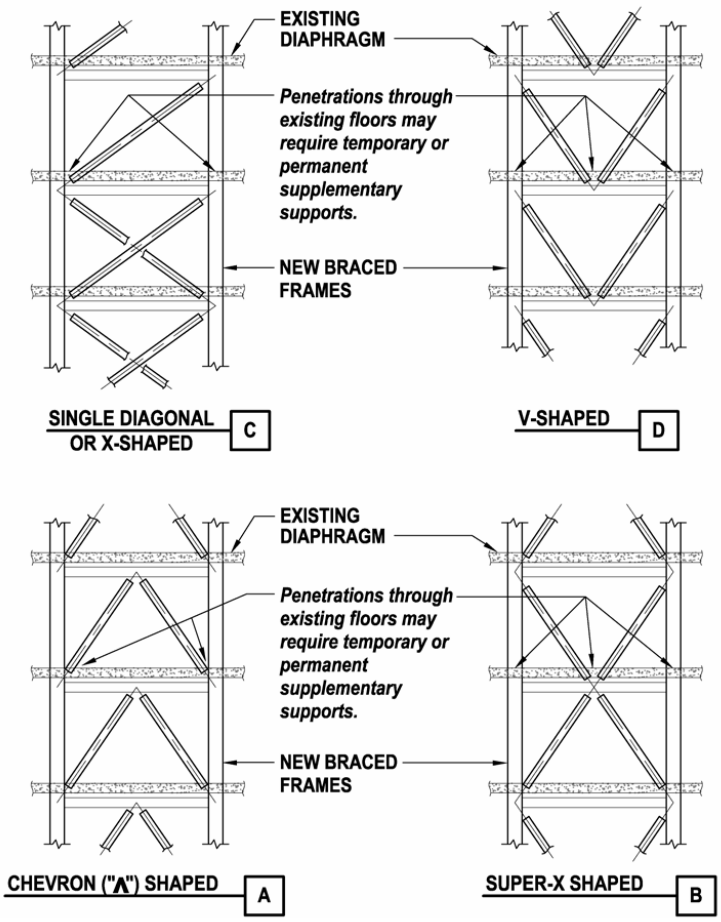


Figure 9.1. Typical braced frame configurations [1].

Braced frame location: the new braces may be located on the exterior or interior of the building. An exterior location generally allows for easier construction access and perhaps less cost, but is visible, exposed to the environment and probably will impact exterior building finishes. Alternatively, exterior bracing may be placed as buttresses, perpendicular to the existing façade. This configuration will probably require more extensive new collectors to deliver lateral forces from the diaphragms but may allow creation of new stair or elevator shafts, or perhaps additional floor area.

The addition of new braced frames to a building will always impact the architectural character and functional uses of the building to some degree. Selection of preferred brace locations must be made considering these issues, such as space layout, corridor locations, doorways, windows, as well as the structural or construction considerations.

A significant concern associated with installing a new steel braced frame in a concrete building is the connection of the beam at the top of the frame in each story to the underside of the existing concrete diaphragm overhead. The connection is generally made by one or more rows of concrete anchors as shown in Figure 9.2. Typically, the anchors are threaded rods set in epoxy, but drilled expansion anchors may be used if they provide sufficient force transfer capacity.
Addition of steel braced frames to an existing building will usually require construction of new footings, or augmentation of existing ones, to resist the concentrated overturning demand. In many cases, the overturning uplift demand will require installation of tie downs. Alternatively, the new frame can be located between two existing column frame lines, instead of directly on or along one frame line, and new foundations or grade beams can be used to engage more than one existing column to resist the uplift demand.

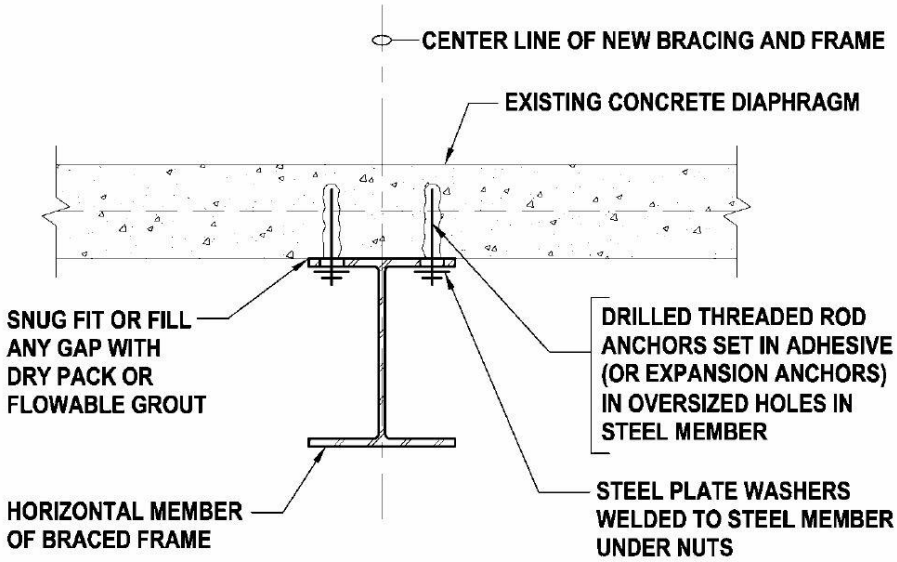


Figure 9.2. Typical connection to concrete diaphragm [1].

Cost/Disruption
The cost and level of disruption associated with installation of steel bracing is generally less than that of shear walls. The number of penetrations that need to be cut through the existing concrete structure and of drilled dowels and anchors may be less than for the shear wall alternative, and the work is generally not as wet or messy. In addition, it will not be necessary to prepare any existing concrete surfaces that will be in contact with new steel members. The new members are discreet and welded or bolted connections are localized. However, there will be noise and vibrations resulting from the required cutting and drilling that will make continued occupancy difficult.

9.2 Add Concrete Shear Wall (Connected to a Concrete Diaphragm)

Addition of shear walls to an existing concrete frame building is a common method of adding significant strength and/or stiffness to the structure. The new walls may be of cast-in-place concrete, shotcrete or fully grouted concrete masonry unit (CMU) construction.
Wall location: the new walls may be placed on the exterior or interior of the building. An exterior location generally allows for easier construction access and perhaps less cost, but is visible, exposed to the environment and may impact exterior building finishes. Walls placed parallel to the façade can

be connected to the exterior edges of floor and roof diaphragms or perimeter concrete frames relatively easily, but will most likely require closure or reduction in size of some windows.
The most significant detail associated with installing a new shear wall in a concrete building is the connection at the top of the new wall to the underside of the existing concrete diaphragm overhead. The construction joint must be made tight, without any gapping, to facilitate transfer of shear forces from the overhead diaphragm into the new wall below and to minimize the possibility of joint slip.

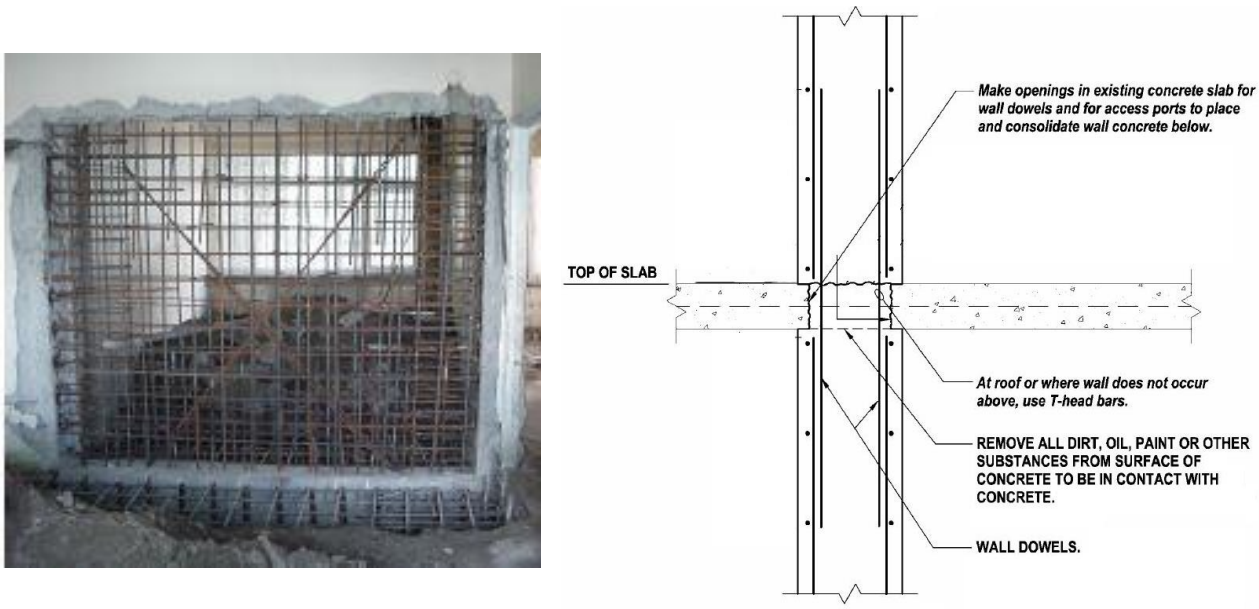


Figure 9.3. New concrete shear wall (left) and concrete wall connection to concrete slab (right).

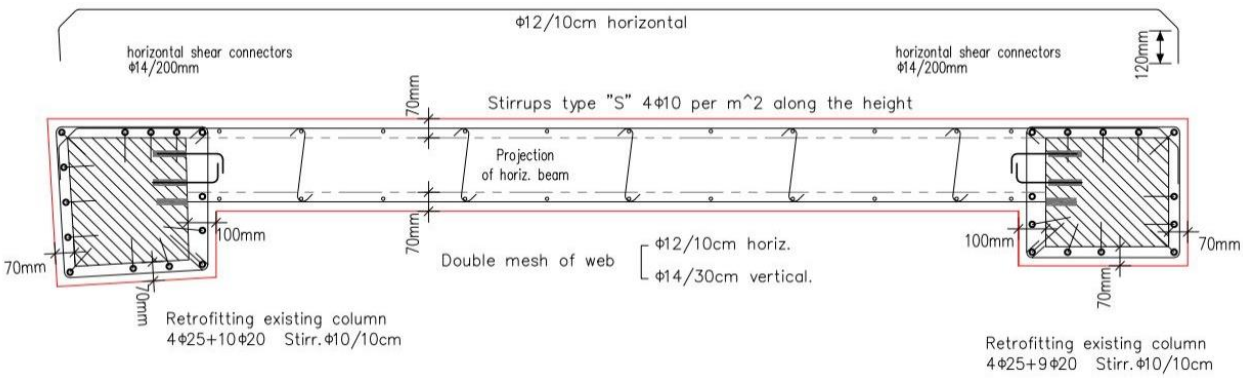


Figure 9.4. Sectional detailing example of new shear wall between existing column frame.

Installation of shear walls in a frame structure, especially in one with a complete frame system, will result in increased diaphragm demands at the individual walls. An advantage of locating the new wall at an existing frame line is that the existing beams can then be used as a collector.
Addition of concrete or masonry shear walls will almost always require construction of new footings, or augmentation of existing ones, to support the added weight as well as to resist the increased and/or concentrated overturning demand.

Cost/Disruption

In general, shotcrete walls are less expensive than cast-in-place concrete because at least one side of the wall forming is eliminated. If shotcrete can be applied against an existing wall at stair, elevator or mechanical shafts, the cost savings of shotcrete is even greater.

Construction of new shear walls in an existing building can be very disruptive to any building occupants. Noise, vibration, and dust associated with many operations, especially cutting holes through and drilling dowels into concrete, can be transmitted throughout a concrete structure.

Placing cast-in-place concrete, shotcrete or even grouted masonry is a wet process and very messy. Shotcreting in an enclosed area creates differential pressures that can spread debris beyond nominal construction barriers.

9.3 Add Masonry Wall (Connected to a Concrete Diaphragm)

Addition of masonry walls to an existing concrete frame building or a masonry building is a common method of adding significant strength and/or stiffness to the structure. Moreover, in cases where walls of partial height are present, the simplest solution is to close the openings by building a wall of full height. This addition of wall for the full height eliminates the short column effect.

The masonry units will typically be constructed up to within one or two courses of the overhead slab ceiling, leaving enough of a gap to allow placement of the upper lift of grout.

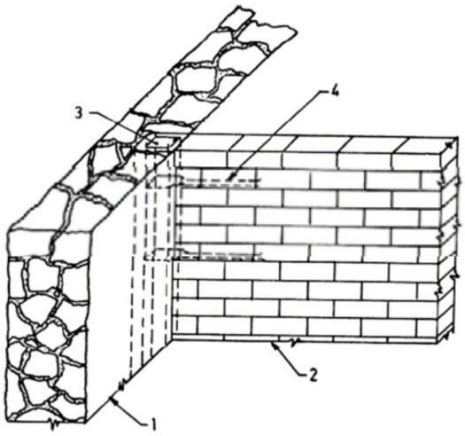
Insertion of cross wall will be necessary for providing transverse supports to longitudinal walls. The new walls can be made of the three different layers usually employed: hollow concrete blocks (100 mm), weak concrete layer (130 mm) and stone (70 mm) as shown in Figure 9.5.

The main problem in such modifications is the connection of new walls with old walls.

Generally, the connection between new and old walls is performed in two ways: T-junction or corner junction. In both cases the link to the old walls is performed by means of a number of keys made in the old walls. Steel is inserted in them and local concrete infilling is made. In the second case, however, connection can be achieved by a number of steel bars inserted in small length drilled holes filled with fresh cement-grout, which substitute keys [14].

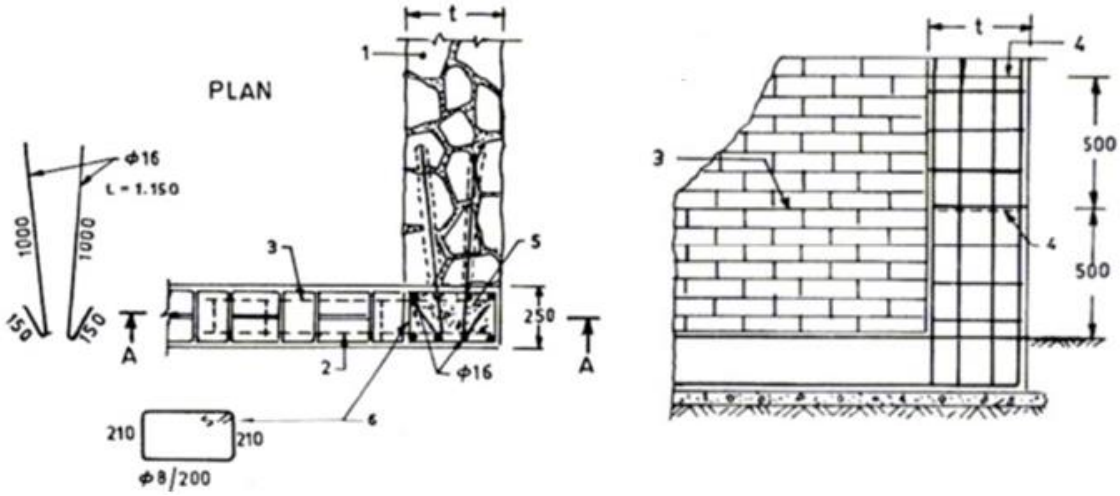
Masonry is usually considered quicker to install and less expensive with respect to the RC wall.

Figure 9.5. Cross section of exterior masonry wall



1. Existing old wall
2. New wall
3. Concrete-column for bonding
4. Connection ties of steel, every fourth course

Figure 9.6. Connection of new brick wall with existing one (T-junction) [10].



Section A-A

1. Existing wall, thickness *t*
2. New wall
3. Horizontal reinforcement with links
4. Connection steel grouted in drilled holes
5. Concrete in column and footing
6. Stirrups

Figure 9.7. Connection of new brick wall with existing one corner junction [10].

9.4 Enhance Column, Beam and Shear Wall with Fibre-Reinforced Polymer Composite Overlay (FRP)

The use of a fibre-reinforced polymer (FRP) overlay with columns has proven to be an efficient rehabilitation technique. Considering that the preferred strength hierarchy for a building type structure is strong-column, weak-beam, it is of utmost importance to strengthen column capacity. For this reason, columns are overlaid with unidirectional fibres in a horizontal orientation, thus providing shear strengthening and confinement similar to that provided by hoops and spirals used with circular columns, and stirrups and ties used with rectangular columns. The confinement enhances the concrete compression characteristics, provides a clamping action to improve lap splice connections, and

provides lateral support for column longitudinal bars. The confinement afforded by this technique does marginally increase the flexural strength and stiffness of the column, but not to the degree of concrete jacketing [15].

FRP composites for structural strengthening are available today in the form of procured strips or uncured sheets. In general, sheets are preferred for improving shear capacity and confinement (fibres parallel to shear forces), while strips are more applicable for flexural capacity strengthening (fibres parallel to longitudinal axis of element).

Table 9.1. Typical properties of fibres for FRP composite materials [16].

Fibre Type	Density	Tensile strength	Young modulus	Ultimate tensile strain	Thermal expansion coefficient	Poisson's coefficient
	(kg/m^3)	(MPa)	(GPa)	(%)	(10^{-6}/°C)	
E-glass	2500	3450	72.4	2.4	5	0.22
S-glass	2500	4580	85.5	3.3	2.9	0.22
Alkali resistant glass	2270	1800-3500	70-76	2.0-3.0	-	-
ECR	2620	3500	80.5	4.6	6	0.22
Carbon (high modulus)	1950	2500-4000	350-650	0.5	-1.2...-0.1	0.20
Carbon (high strength)	1750	3500	240	1.1	-0.6...-0.2	0.20
Aramid (Kevlar 29)	1440	2760	62	4.4	-2.0 longitudinal 59 radial	0.35
Aramid (Kevlar 49)	1440	3620	124	2.2	-2.0 longitudinal 59 radial	0.35
Aramid (Kevlar 149)	1440	3450	175	1.4	-2.0 longitudinal 59 radial	0.35
Aramid (Technora H)	1390	3000	70	4.4	-6.0 longitudinal 59 radial	0.35
Aramid (SVM)	1430	3800-4200	130	3.5	-	-
Basalt (Albarrie)	2800	4840	89	3.1	8	-

Table 9.2. Typical properties of matrices used in FRP composite materials [16].

Property	*Matrix*		
	Polyester	Epoxy	Vinyl ester
Density (kg/m^3)	1200 - 1400	1200 - 1400	1150 - 1350
Tensile strength (MPa)	34.5 - 104	55 - 130	73 – 81
Longitudinal modulus (GPa)	2.1 – 3.45	2.75 – 4.10	3.0 – 3.5
Poisson's coefficient	0.35 – 0.39	0.38 – 0.40	0.36 – 0.39
Thermal expansion coefficient (10^{-6}/°C)	55 - 100	45 - 65	50 - 75
Moisture content (%)	0.15 – 0.60	0.08 – 0.15	0.14 – 0.30

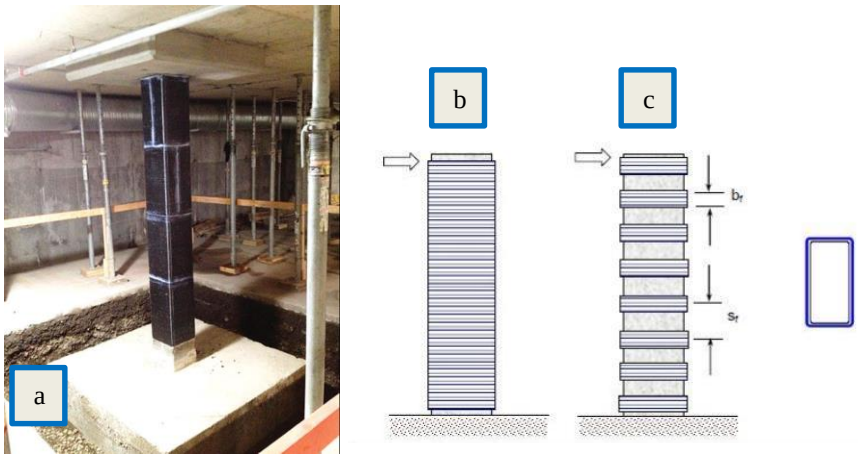


Figure 9.8. Shear strengthening of column with carbon - FRP composite materials: (a,b) sheets; (c) strips [20].

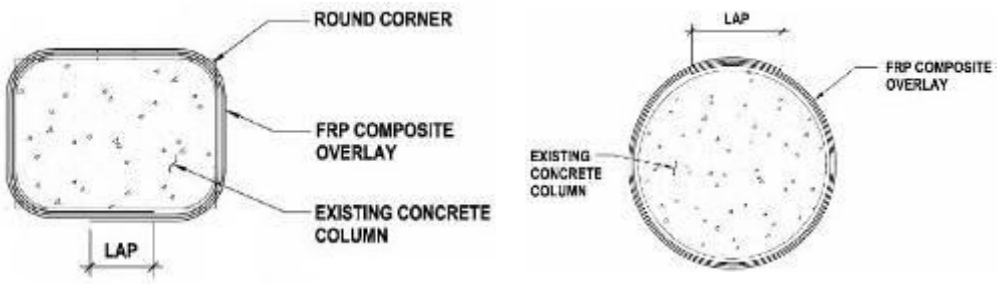


Figure 9.9. Rectangular and circular column jacketing with FRP composites [1].

For the two-sided or three-sided jacket, failure occurs due to early de-bonding. For the closed jackets cases, failure occurs due to fibre-fracture under tension or shear (corners). The applications illustrated in Figure 9.8 b and c should be avoided. The retrofitting is best when the jackets are fully anchored (Figure 9.10 f, g), since the composite material is able to reach much larger values of strain without bondage early failure. More details regarding the proper anchorage of the FRP sheets can be found in Jinno *et al*. [17], Karantzikis *et al.* [18] and Kobayashi *et al.* [19].

Shear failure is catastrophic and occurs usually without advance warning. Thus, it is desirable that the beam fails in flexure rather than in shear. Deficiencies for shear occur for several reasons, including insufficient shear reinforcement or reduction in steel area because of corrosion, increased service load, and construction defects. To increase the shear resistance of concrete beams, sheets and laminates of FRP are generally applied on the faces of the elements to be strengthened.

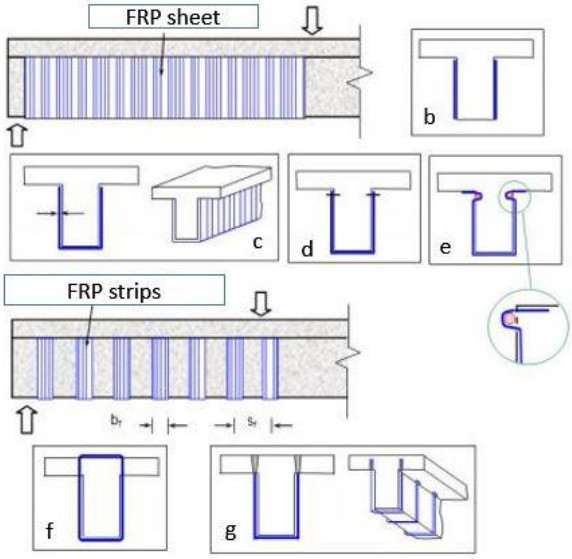


Figure 9.10. Representative configurations of FRP shear strengthening of RC beams [20].

Figure 9.11. Strengthening the flexural capacity of RC beam with carbon-FRP strips.

FRP composites are also used for retrofitting existing RC shear wall, similarly to the way they are applied for columns. The FRP overlay is a technique that is used to enhance the in-plane shear capacity of the shear wall [21]. The overlay can be applied to one or both sides of a wall, and where possible, should wrap around the ends of the wall to aid in anchoring the overlay. The rehabilitation technique is bond-critical, regardless of whether or not the material wraps around the end of the walls. Uni-directional (horizontal-oriented) fibres are used to enhance the shear capacity, creating a predominately flexural post-yield response. Vertically-oriented fibres in bidirectional layouts will limit the vertical strains to that of the FRP composite, inhibiting the ductile behavior. Therefore, the rehabilitation technique for walls is limited to horizontally oriented fibres, unless there are extenuating circumstances. The shear enhancement may change the wall's response from a shear-dominated behavior to a flexural, sliding shear, or rocking behavior. See Figure 9.12 for example of wall layouts. Special attention is required for L or C shaped shear walls, since early de-bonding is quite prone to happen in the internal corners. To address that, the common practice is to put FRP

anchors every 200mm along the height of the corner. More information about that topic can be found in Hiotakis *et al.*[22], Lombard *et al.*[23], Karantzikis *et al.* [24].

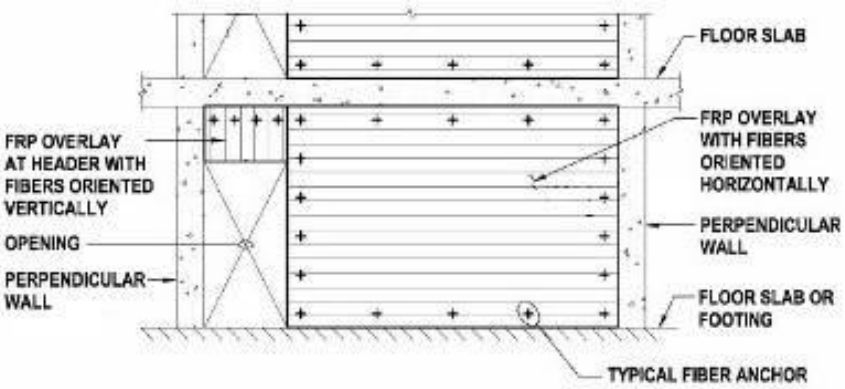


Figure 9.12. Strengthening of concrete shear walls using FRP composite [1].

Cost/Disruption

To appropriately evaluate the cost of a retrofit scheme using and FRP overlay in comparison to traditional retrofit concepts (such as concrete or steel jacketing), one needs to consider the cost of the raw material, the level of specialization required by the contractor to install the system, the cost of labour and equipment, the cost of quality control and quality assurance, the temporary impact of disruption during construction, and the permanent impact to the building functions.

Although FRP overlays are relatively expensive compared to steel and concrete, they can offer advantages when only limited access is available or minimal disruption of existing conditions is desired.

Table 9.3. Advantages and disadvantages of FRP reinforcement

Advantages	Disadvantages
Higher ratio of strength to self weight (10 to 15 times greater than steel)	Higher raw material cost
Carbon and aramid fibre reinforcement have excellent fatigue characteristics	Lower elastic modules (except some Carbon FRPs)
Excellent corrosion resistance and electromagnetic neutrality	Glass FRP reinforcement suffers from stress corrosion
Low axial coefficient of thermal expansion	Lack of ductility
Fast-track application	Low fire resistance
Insignificant increase of member-dimensions	Highly trained workforce

9.5 Enhance Concrete Column, Beam and Shear Walls with Concrete or Steel Overlay

Adding a fibre reinforced polymer (FRP) composite overlay to a concrete column or beam is a recent approach to addressing seismic deficiencies. Adding a concrete or steel jacket is a more traditional method of enhancing a deficient concrete column [25]. Figure 9.13 provides examples of concrete and steel jacketing for a rectangular column.

For what concern coupling beams, they need a large amount of shear reinforcement in the plane of coupling the shear walls and the coupling beams, to dispel the seismic energy produced by the earthquakes. To surpass this amount of reinforcement an auxiliary technique is proposed, that is, diagonally reinforced coupling beams. Diagonal reinforcement usually consists of minimum four bars per diagonal.

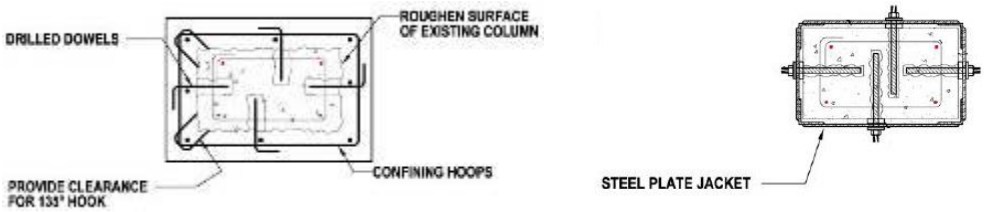


Figure 9.13. Concrete jacket (left) and steel jacket (right) [1].

Concrete overlays: concrete jacketing will take up a larger cross section than either FRP overlays or steel overlays. The surface of the existing concrete must be roughened appropriately.
Reinforcing steel will need to be in at least two pieces to get it around the existing column. 135° hooks are required for confining ties and may dictate the size of the overlay in order to provide enough room for the hook extension.
Steel jackets: steel jackets require at least two pieces to get around the existing column and involve field welding. Like FRP overlays, when the aspect ratio of a rectangular column gets too large, the jacket becomes less effective.

Figure 9.14. Column jacket examples: steel (left) and RC (right).

Figure 9.15. Beam RC jacket application examples.

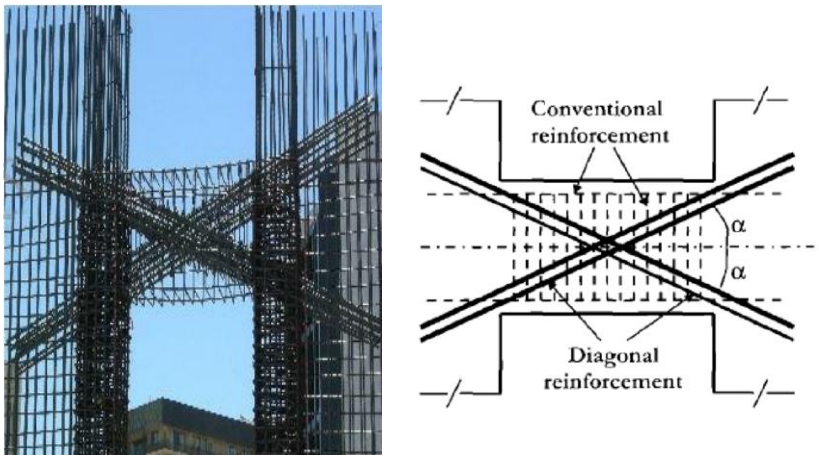


Figure 9.16. Diagonal reinforcement in RC coupling beams.

The technique is also used for eliminate the short column mechanism, exactly strengthening the column with RC or steel jacketing. FRP jacketing can be used as well, for more details Cf. §9.4. The special confining reinforcement with closely spaced closed stirrups (*e.g.* Φ10/100 in *mm*) must extend beyond the short column into the columns vertically above and below by a certain distance as shown in Figure 9.17. Beams short in length have similar problems; again diagonal reinforcement and small stirrups spacing is a possible solution.

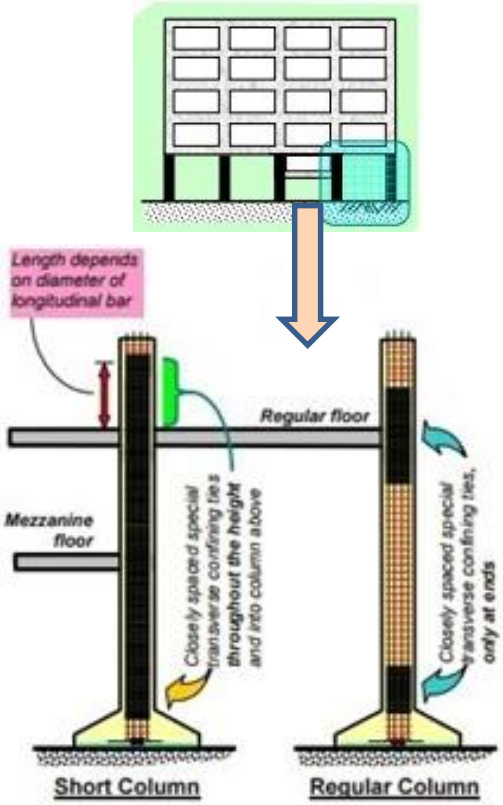


Figure 9.17. Detailing of sectional reinforcement of RC short columns [4].

9.6 Brace or Remove URM Parapet

Past earthquakes have consistently shown that parapets are the first elements to fail in earthquakes due to inadequate bending strength and ductility. Parapets tend to have greater damage at mid-span of diaphragms due to higher accelerations and displacements from the oscillating diaphragm.

RM parapets can be braced or removed to minimize the falling hazard risk. Bracing is usually done with a steel angle brace. The brace is anchored near the top of the parapet and to the roof.

The existing roof framing may need localized strengthening to take the reaction from the brace. Roof-to-wall tension anchors are typically part of parapet bracing. See Figure 9.18 for an example of parapet bracing. If the top of the parapet is removed, the vertical compressive stress on roof-to-wall anchors is reduced, so removing the parapet is often combined with adding a concrete cap or bond beam as part of the roof-to-wall anchorage [26].

Cost/Disruption

Adding parapet bracing and roof-to-wall tension anchors provide some of the most effective seismic rehabilitation for reducing life safety risks. Disruption is typically relatively low since occupants can remain in place. Disruption can increase noticeably if the roof has to be removed for installation.

Figure 9.18. Parapet bracing [26].

9.7 Add Out-of-Plane Bracing for URM Walls

Two types of bracing can be used: diagonal braces that reduce the effective height of the masonry wall and vertical braces or strongbacks that span the full height of the inside face of the wall (Figure 9.19). Vertical braces can be surface mounted or, when aesthetic considerations are paramount, recessed into the wall. Braces are typically done with steel, but strongbacks can also be done with wood posts or with concrete pilasters [27].

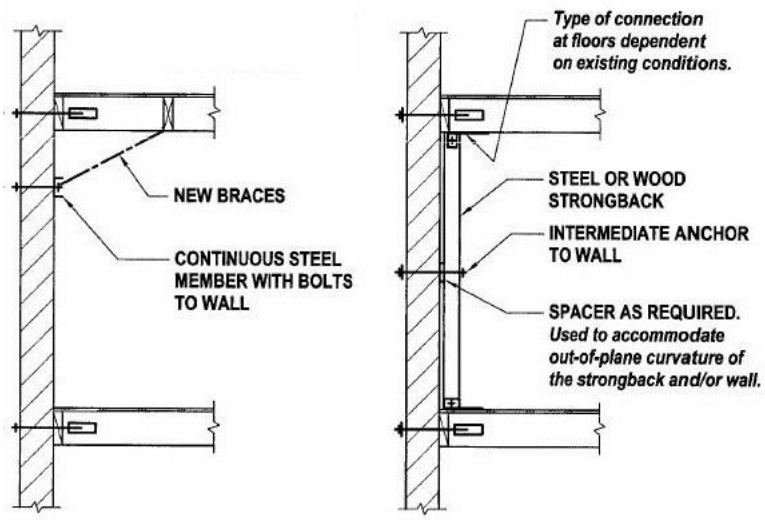


Figure 9.19. Wall bracing: diagonal (left) and vertical (right) [26].

Cost/Disruption

Diagonal bracing is usually less expensive, but is considered less reliable than vertical bracing. Exposed braces are typically less expensive than more architecturally sensitive alternatives like recessed vertical braces or reinforced cores. Installation of bracing is fairly disruptive since it must occur around the entire perimeter; and it involves drilled dowels, and accessing and connecting to horizontal diaphragms.]

9.8 Add Reinforced Cores to URM Walls

Installing the reinforced core involves drilling a core from the roof down the inside of an unreinforced masonry wall. A steel reinforcing bar and grout are placed inside the hole to increase the wall strength. This process is used to avoid the aesthetic impact of exposed bracing described in Section 9.12.

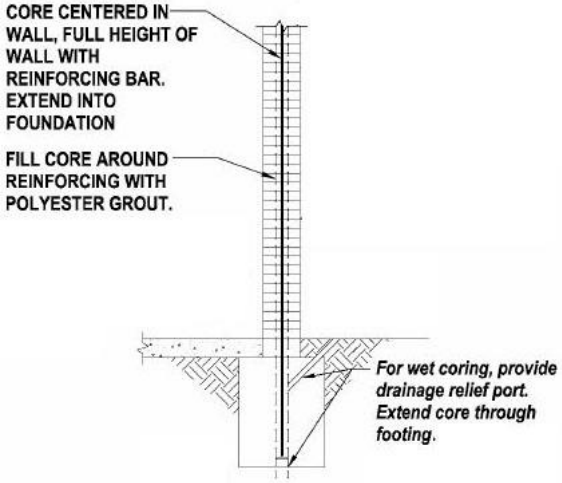


Figure 9.20. Reinforced Cores [1].

Cost/Disruption

Adding reinforced cores can be considerably more expensive than exposed bracing, so it is usually only performed in historically and architecturally sensitive buildings. Since the core is placed inside the wall, the disruption to interior and exterior faces is limited to sealing cracks, access to the roof to place the drilling equipment, and drilling noise and vibration.

9.9 Add Concrete Overlay to Masonry Wall

New concrete is applied against an existing unreinforced masonry wall to increase the shear capacity of the wall. The new concrete is attached to the old wall with adhesive anchors and can either be cast-in-place concrete or sprayed-in-place. In rehabilitation work, sprayed concrete, known as shotcrete, is more commonly employed than cast-in-place construction, since the existing wall provides the back-side form. The thickness of the new concrete varies with strength requirements, but it is usually from 10 to 30 cm.

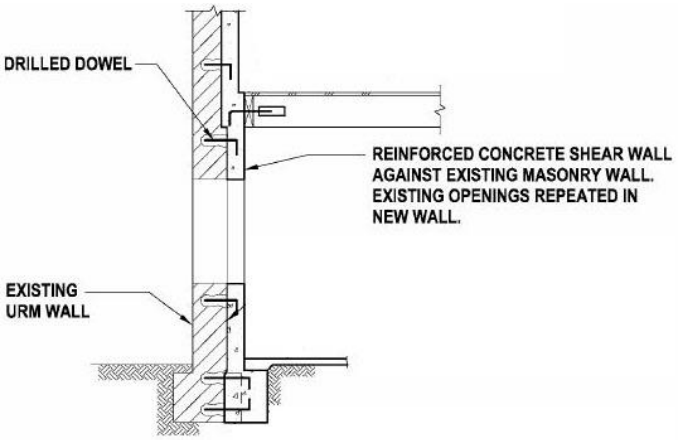


Figure 9.21. Concrete or shotcrete wall overlay [1].

Cost/Disruption

Adding new concrete, particularly with shotcrete, can be quite disruptive. Where access is sufficient, shotcrete is typically chosen as it is less expensive than cast-in-place work which requires front-side formwork.

Shotcrete: Placing shotcrete requires access for the hose and concrete truck and sufficient room to spray the concrete. It is desirable to shoot downward, so scaffolding is needed at upper portions of walls to achieve the necessary angle. Spraying is noisy and very dusty. If an indoor wall is being shot, the room will usually be sealed off with plastic sheeting to control dust. During shooting, residue – known as rebound – forms at the base of the shoot and must be cleaned away so that it does not become part of the overlay. Protection against rebound on existing floor and wall surfaces is needed.

Cast-in-place concrete: Placing cast-in-place concrete also requires access for the hose and concrete truck. Less front-side access is needed than shotcrete, but a front-side form is required with the associated sawing and hammering noise of construction. Concrete placement is noisy, and in addition to workmen and concrete truck noise, there is the vibrator used to consolidate the concrete.

9.10 Add Fibre-Reinforced Polymer Overlay to Masonry Wall

An FRP overlay, typically made of glass or carbon fibres in an adhesive matrix, is applied against an existing unreinforced masonry wall to increase the shear strength of the wall [28]. The existing wall surface must be prepared to receive the new material, and after application the fibre composite must be protected against ultraviolet rays. Overlay with vertical strips are used when only improvement to out-of-plane resistance is needed. Diagonal strips have been used to resist diagonal tension stresses from in-plane shear (Cf. §9.4 for more details about the material properties of FRP composites).

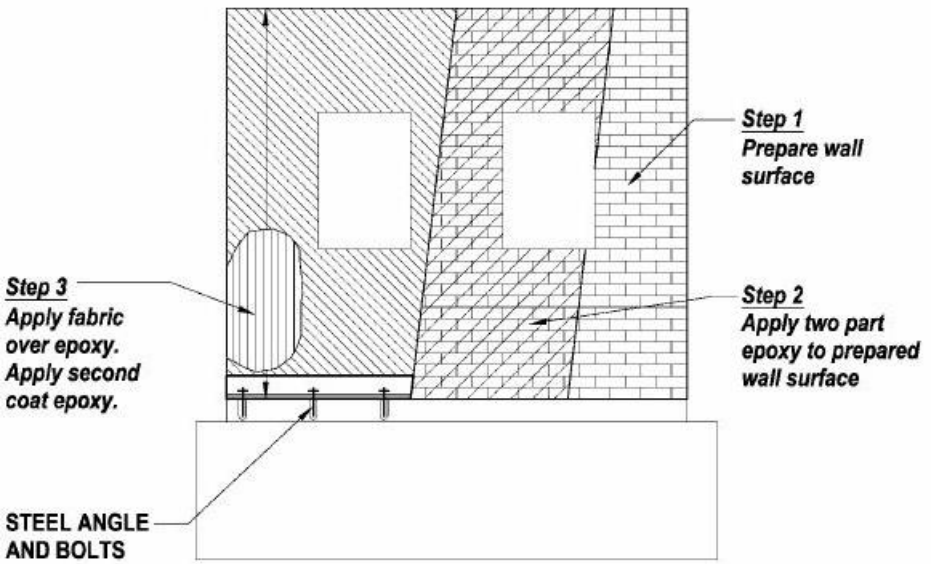


Figure 9.22. Fiber composite wall overlay on URM wall.

Cost/Disruption

Fibre composites are relatively expensive as an application for masonry wall strengthening and have not seen significant use. Disruption comes from the sandblasting and surface preparation of the masonry wall, the fumes from the adhesives used in application of the fibre composite and removal and access requirements where the fibre transitions from story to story at the floor levels.

9.11 Infill Opening in a URM Wall

Window and door openings are filled to increase the shear capacity and reduce the shear stresses on the unreinforced masonry wall. The opening is typically filled with concrete, reinforced concrete masonry units, or reinforced clay brick, rather than with unreinforced masonry due to code concerns with adding unreinforced masonry. To provide adequate shear transfer between the existing wall and the new infill, the interface can be toothed, but more typically, drilled dowels are used.

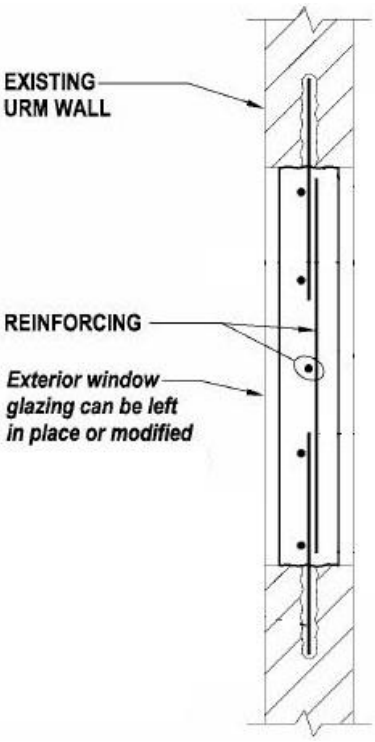


Figure 9.23. Infilling an opening in a URM wall [1].

Cost/Disruption

Infilling an opening is relatively inexpensive if no architectural treatment is done to the face. Disruption is also more localized compared to other in-plane wall strengthening methods like concrete and fibre reinforced polymer overlays. Noise will occur during drilling holes for drilled dowels and placing the infill.

9.12 Add Supplemental Vertical Support for Truss or Girder

A steel or wood post is added under existing trusses and girders. Steel members are smaller; wood members are less expensive. For certain architectural approaches, leaving the supplemental posts bare is compatible with the existing aesthetic. If it is not, the posts can be furred at added cost.

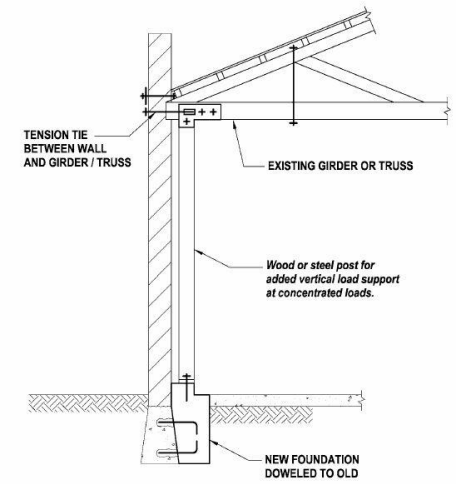


Figure 9.24. Supplemental vertical support [1].

Cost/Disruption

The relative cost of adding supplemental supports depends on the number used, whether they continue down to the ground and whether a new foundation is installed. Interior occupants will be disrupted locally as the posts are installed, and the usable space in the vicinity of the posts will be reduced.

9.13 Add Veneer Ties in a URM Wall

If sufficient metal veneer ties are not present to anchor the veneer, new ties can be provided. Figure 9.25 shows anchorage using drilled dowels to connect the withes from the interior in a typical brick wall.

The most reliable location of the drilled dowel is in the centre of the brick, but this will cause the largest aesthetic impact. The dowel can be placed in the bed joist or bed and head joint intersection to minimize the impact.

Brick veneer vs. stone veneer: Anchoring thick stone is usually easier to do from the interior because the thickness of the stone permits greater cover on the face, reducing the likelihood of spalling.

Corrosion considerations: Any dowel installed from the exterior should be done in stainless steel to minimize corrosion.

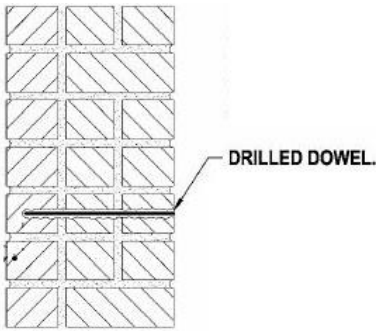


Figure 9.25. Veneer ties in brick masonry [1].

Cost/Disruption
Veneer anchorage can be relatively expensive depending on the number of new ties added. Ties installed from the inside are much more disruptive.

9.14 Infill Opening in a Concrete Diaphragm

Addition of a structural infill to close an existing opening is a relatively simple method of correcting this type of local diaphragm deficiency in a concrete diaphragm. The new infill will reduce concentrated shear and chord force demand in the surrounding diaphragm and eliminate the need for often non-existent local chords around the edges of the opening. In almost all cases, the new infill will be made with cast-in-place reinforced concrete or shotcrete. While it is conceivable, and perhaps possible in some unusual cases, to close the opening with steel plate or a precast concrete "plug," the connections to the surrounding slab are very problematic, and their effectiveness as a mitigation measure is doubtful.

Connection to existing concrete floor and roof diaphragms: sufficient dowels must be placed into the existing diaphragm slab on all sides of the opening to transfer the required shear demand to and from the infill section. Forms may be supported from the floor below or suspended from the surrounding floor or roof. This latter option is much more common for smaller openings or for openings surrounded by waffle ribs, pan joists or beams. Since the concrete infill will shrink relative to the surrounding slab, some care should be given to use shrinkage compensated mix.

Cost/Disruption
The cost of this type of infill is very modest and will generally be a very small component in the overall retrofit project. Except for the noise and vibration associated with the dowel drilling, disruptions associated with this type of infill will be very localized, affecting only the immediate surrounding floor area and the area on the floor below.

9.15 Add RC Overlay to a Concrete Diaphragm

The major deficiencies found in the concrete diaphragm are inadequate in-plane shear capacity of the concrete diaphragm and inadequate flexural capacity. In strengthening shear capacity, overlaying the existing concrete diaphragm with new reinforced concrete topping slab and reducing the shear by providing supplemental vertical-resisting elements will improve the capacity. In strengthening the flexural capacity, casting a new chord member integral with the slab after removing the edge of diaphragm slab and reducing the flexural stress by providing additional vertical-resisting elements will improve the flexural capacity.

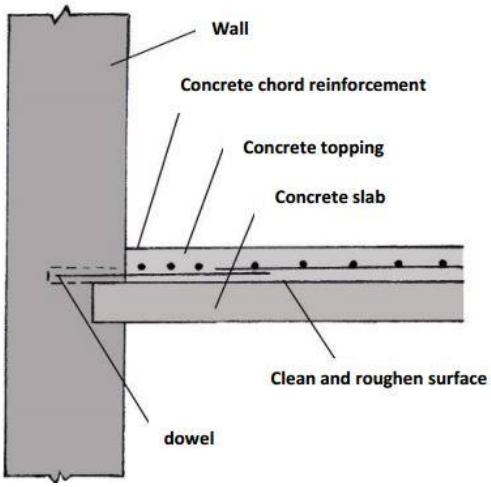


Figure 9.26. RC topping slab and chord on existing concrete diaphragm.

9.16 Add Fibre-Reinforced Polymer Composite Overlay to a Concrete Diaphragm

The use of an FRP overlay with slabs for in-plane shear strength (diaphragm shear) enhancement is a very new technique that has had limited implementation. For shear enhancement of monolithic slab construction, the fibres are oriented parallel to the applied shear direction. Joint strengthening usually employs bi-directional fibres orientated at 45° to the shear plane.

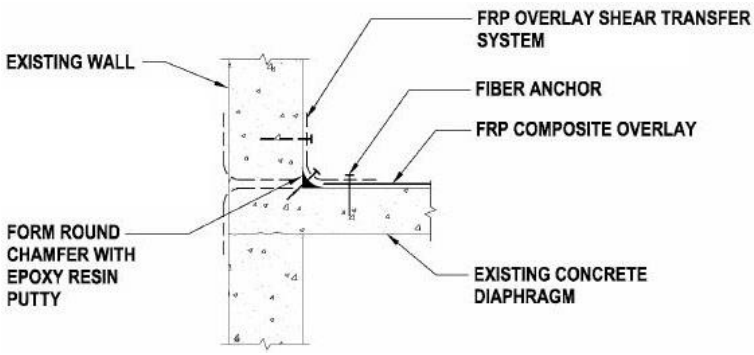


Figure 9.27. Shear strengthening of concrete diaphragm using FRP composite [1].

9.17 Add Horizontal Braced Frame as a Diaphragm

Providing a horizontal braced frame as a diaphragm strengthening technique is useful if the existing floor cannot be disturbed for functional reasons or the cost of replacing the existing diaphragm is more expensive (*e.g.*, a sloped roof). This is also an alternative when concrete overlays add too much mass or lead to other construction complications. The existing diaphragm could be constructed of concrete filled or unfilled metal deck. The new horizontal bracing is added under the existing diaphragm, in which the existing framing with new diagonal members forms the horizontal bracing system. The diaphragm shears are shared with the existing diaphragm in proportion to the relative rigidity of the two systems. The design philosophy is generally to have the diaphragm remain essentially elastic, with the goal of achieving ductile inelastic behaviour in the vertical lateral force-resisting elements.

Cost/Disruption

These costs of adding horizontal bracing must be weighed against that of a concrete overlay. Temporary removal or relocation of nonstructural elements such as piping and partition walls are required and should be included in the cost evaluation for both options. The horizontal braced frame requires connection modifications, which are locally very disruptive.

9.18 Add Seismic Joint

Expansion joints are created to transform a single irregular building into multiple regular structures, permitting independent movement during dynamic response.

Several locations should be considered when placing an expansion joint. Locations where differential movement is likely are prime locations, such as at the connection between a building addition and the original construction, or where different materials come together. Other preferred locations are areas that feature a change or a turn in geometry, such as in the reentrant corner of an L-, U- or T-shaped building.

To design a separation joint, the maximum drift of the two units must be calculated by the structural consultant.

9.19 Improve Slab-Column Shear Transfer

Increasing the depth of the slab locally around the column improves the punching shear capacity of the slab and it is an easy way of dealing with the absence of beam.

Figure 9.28. Retrofitting measures protecting slab from punching shear: steel (left) and RC (right) applications.

10 IMPLEMENTATION IN PALESTINE

The retrofitting techniques proposed in Table 5.1, Table 6.1, Table 7.1 and Table 8.1represent the current state-of-the-art. Implementing some of these measures in the Palestinian region may be challenging for the local engineers, constructors and workers. This chapter presents the practical limitations that a retrofitting study in Palestine should take into consideration.

- Some schemes demand complex structural modelling (isolation, damping, seismic joint) and sophisticated engineering software that might not be widely accessible.
- The availability of some materials (i.e. FRP, steel hollow sections or bolts, dampers, tin rubber bearings) is also questionable due to importing restrictions. Steel constructions with bolts (i.e. X-braces) also require high level of accuracy in the dimensions/holes that perhaps is hard to meet.
- The level of training that the workers must have to be able to apply the measures is also an important consideration; the welding of steel and the application of FRP composites for example, require special training sessions for workers and meticulous supervision from the civil engineer.

Traditionally the common construction practice involves mainly RC and masonry elements. These materials are easy to find and less expensive than FRP, structural steel and rubber (isolation).

Table 10.1. Evaluation of applicability of proposed techniques

Retrofitting schemes	Availability of the Material	Familiarity	Low Demand for Specific Training	Simplicity of Structural Analysis
RC	✓✓✓	✓✓✓	✓✓	✓✓
Steel	✓✓	✓✓	✓	✓✓
Masonry	✓✓✓	✓✓✓	✓✓✓	✓✓
FRP	✓	✓	✓	✓✓
Seismic Joint	✓✓✓	✓	✓✓	✓✓
Seismic Isolation	✓	✓	✓	✓
Supplemental Damping	✓	✓	✓	✓

New masonry or RC shear walls and RC jacketing of existing elements seem to be the cheapest and most easily applicable retrofitting technique for the region. The constructors, although familiar with steel reinforcement, still hesitate to widely use structural steel; that might change in the near future. The popularity of FRP composites, although it increases worldwide, is still very low in Palestine, for various reasons (importing issues, training of workforce, familiarity). Finally, schemes like seismic isolation and supplemental damping are realistically unlikely to be implemented.

11 ANNEX 1

This section of the report is mainly focused on foundation rehabilitation. Due to the fact that the proposed techniques are cross-cutting and common for all the typologies, retrofit measures are presented as annex.

Foundation analysis can be one of the most challenging areas of seismic rehabilitation. Different assumptions regarding base conditions of restraint, soil properties, and locations and types of potential nonlinearity can lead to widely varying results. For many buildings, it can take significant analytical effort in modelling and evaluating results to understand how the foundation interacts with the superstructure and surrounding soil under earthquake loading.

When careful analysis reveals that new foundations must be added or that existing foundations must be enhanced, the structural engineer must have a good understanding of soil engineering issues; rehabilitation goals, performance criteria, and assumptions; and construction techniques and limitations. Because of the cost of foundation rehabilitation, other options should be fully explored, and the need for foundation modification should be thoroughly investigated.

11.1 New Foundations

11.1.1 Add Shallow Foundation Next to Existing Shallow Foundation

When a concrete overlay is placed against an existing wall, a new footing is typically needed. A common situation is the existing footing is a continuous strip footing and the new footing is either a strip footing or a grade beam. Figure 11.1 shows an example of a new concrete wall and footing against and existing unreinforced masonry wall and concrete strip footing.

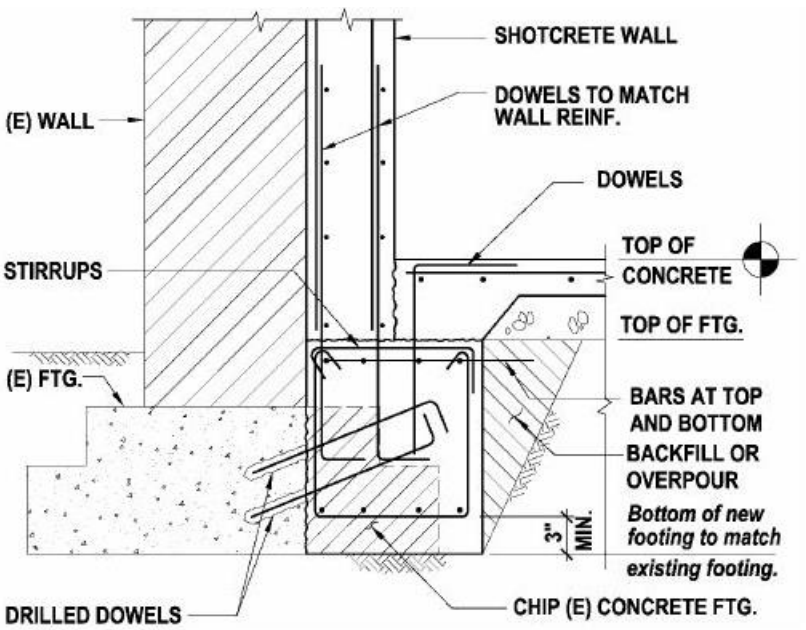


Figure 11.1. New concrete strip footing next to existing strip footing [1].

It is standard practice to connect the new and existing footings with drilled dowels. Dowels in the footing and wall above should be designed to be sufficient to transfer the force intended to be resisted under the existing footing.

If the existing footing is a wide unreinforced masonry or poorly reinforced concrete beam, the bottom drilled dowels can be extended deep into the existing footing near the base of the footing to serve as positive reinforcing.
Another important feature is that, if possible, the new footing does not need to be deeper than existing one to avoid movement and settlement under the footing leading to damages. Generally, it is common to extend the bottom of the new footing down to match the depth of existing one.

Cost/Disruption
Adding a new footing is quite disruptive and costly. The existing slab-on-grade must be saw cut and removed, then the trench excavated, drilled dowels installed, rebar laid, debris in the footing removed and concrete placed. This is all time-consuming, messy and noisy.

11.1.2 Add Deep Foundation Next to Existing Shallow Foundation
Adding a new deep foundation next to an existing foundation is occasionally done, such as drilled piers under a new wall next to an existing strip footing. Drilling limitations can be significant and they include access requirements for the drill rig, height restrictions for the drill rig, the offset needed to get the edge of the drill up against the existing wall, vibration during drilling and utilities in the way of the drilling.
Usually, the drilled piers are spaced at a sufficient distance that the existing footing and walls can span around or over the open hole.

11.2 Structural Rehabilitation for Existing Shallow Foundations
11.2.1 Add micropiles
To improve the compression and/or tension capacity of the existing footing, the footing is widened and micropiles are added (Figure 11.2).
When micropiles are added together with the strip footing, resistance is shared between the two different elements, depending on their relative rigidity. The connection to the new footing is performed with bars drilled through the existing footing [29].

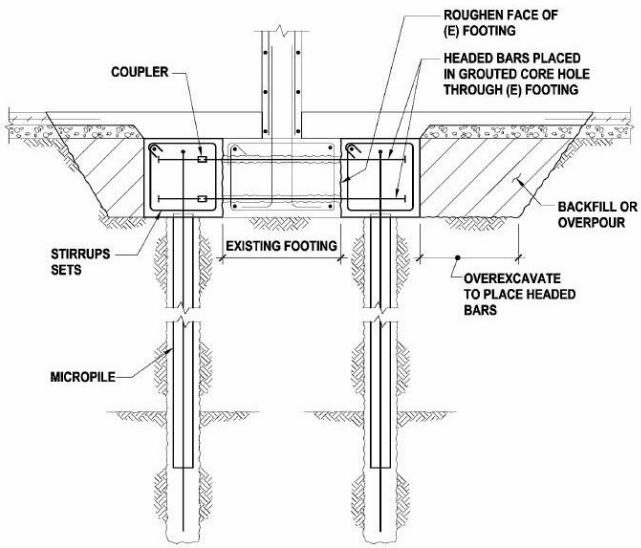


Figure 11.2. Section of micropile enhancement to existing strip footing [1].

Cost/Disruption
Micropiles are typically less expensive than drilled piers, unless very large capacities are required. Excavation noise and dust, and drilling and grouting noise must be considered as part of the rehabilitation strategy.

11.2.2 Enlarge existing spread footing

An existing spread footing may be under RC moment frame or a concrete column below a discontinuous shear wall and be subjected to compression or tension forces that exceed the footing capacity. The existing footing can be enlarged or replaced to increase compression capacity or the dead load for resisting tension.
Existing reinforcing should be preserved in the footing. This will typically require placing new drilled dowels at a higher elevation, with a resulting lower moment capacity.

Figure 11.3. Enlarged footing example.

Transferring shear between the existing footings is necessary. This can be accomplished by roughening the existing footing face. A procedure could be digging dig the new footing slightly deeper than the existing footing and undercut the soil at the edge of the existing footing, so that the new footing acts as a corbel to resist downward pressure from the existing footing.

Cost/Disruption
Enlarging or replacing an existing footing is a localized but disruptive process, involving excavation, dust, mud, drilling noise and concrete placement. Protection of existing finishes in the vicinity and in the working path is necessary.

11.3 Structural Rehabilitation for Existing Deep Foundations

11.3.1 Add a Mat Foundation, Extended Pile Cap or Grade Beam

The deficiency addressed by this technique is inadequate compression capacity of an existing deep foundation element. The technique involves taking advantage of the contributions of shallow foundation elements that are part of the overall foundation system.
When existing piers or piles have inadequate capacity, the usual approach to increasing their capacity is to install new micropiles or piers connected by grade beam to the existing adjacent piles or piers. Figure 11.4 shows an example of existing piers whose capacities are augmented by installing a mat between the piers.

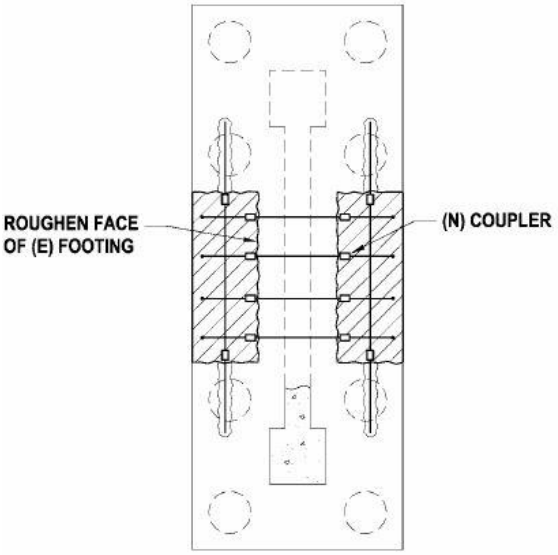


Figure 11.4. New mat foundation between existing drilled piers (plan) [1].

Cost/Disruption

Adding a new mat or cap or grade beam is quite disruptive and costly. The existing slab-on-grade must be saw cut and removed, then the foundation excavation completed, drilled dowels installed, rebar laid, debris in the excavation removed and concrete placed. This is all time-consuming, messy and noisy.

11.4 Ground Improvement for Existing Shallow and Deep Foundations

Typical goals for ground improvement under existing shallow and deep foundations can be classified into two categories: mitigating the potential impacts of an identified geologic hazard, and enhancing the capacity of the foundation by changing the load-deformation characteristics of the foundation soil.

11.4.1 Compaction Grouting

Compaction grouting involves the injection of a very stiff grout at a high pressure into a layer of soil to force the individual soil particles into a tighter packing [30]. The resulting increase in the density of the soil substantially increases its resistance to liquefaction as well as its bearing capacity. Compaction grouting can be performed in a wider range of soil types than other grouting methods. It can be performed in various types of sands, and clayey materials, but has limited effectiveness in clean coarse sands and gravels and in high plasticity soils.

The grout is required to have low flowability. This low flowability is necessary because the most important characteristic for effective densification is for the grout to form a controlled mass, which is columnar or tear-shaped, when injected. If it behaves instead like a fluid in the ground, it can create fractures in the soil, through which the grout can flow. Since the effectiveness of the grout is based on its ability to stay as a mass pushing soil particles together, that effectiveness is lost when the grout flows.

The grout – which consists of mostly of sand, cement, and water – is injected through grout holes that are drilled in a grid pattern of between 1.20 and 4 meter. Grout is usually injected in stages. Staging involves the injection of only a few meter of grout hole at a time. Staging can proceed from top-down or bottom-up, the latter approach being the most commonly used.

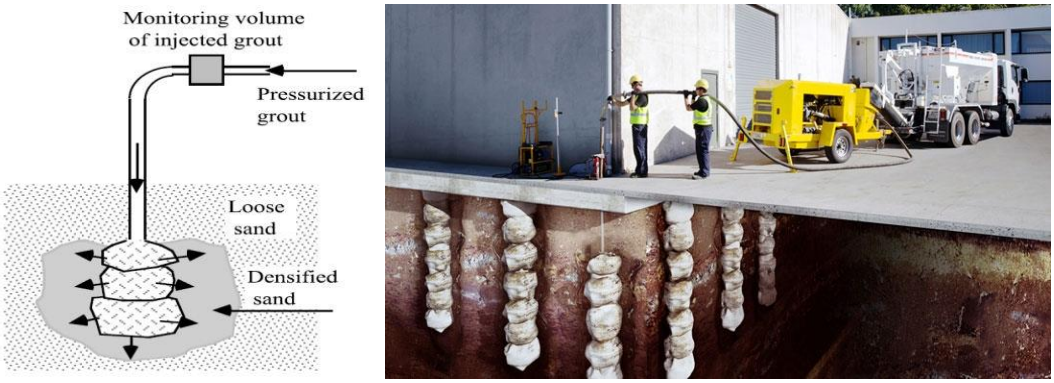


Figure 11.5. Compaction grouting example.

Cost/Disruption

Compaction grouting is quite disruptive and costly especially if the creation of injection holes includes drilling through existing pile or pier caps, grade beams, or concrete footings and slab.

This grouting process could also be time-consuming and messy. Disruption to the current operations of the building is usually minimized by performing the compaction grouting at night and cleaning up the work area before the start of work the next morning. Compaction grouting is generally less costly than permeation grouting for a given scope of work.

11.4.2 Permeation Grouting Under Existing Shallow and Deep Foundations

Permeation grouting involves the injection of chemical or cement grout into the pore spaces of soils and aggregates without displacing the materials. This helps solidify the usually sandy soils that are amenable to this technique. The resulting increase in shear strength of the soil substantially increases its resistance to liquefaction as well as its bearing capacity. Permeation grouting can be performed in sands and sandy soils that contain minor amounts of fine particles. The structure and the size of voids in the soil structure dictate the type of grout that can be effectively used. In general, either micro-fine cement grout or a chemical grout.

The grout is injected through grout holes that are drilled in a grid pattern of between 0.5 and meter. Grout is usually injected in a strict primary-secondary pattern. Alternate primary holes are drilled and grouted first followed by the secondary holes. The level of solidification achieved is verified by exhuming grouted soil bulbs, taking samples of the grouted soil and performing unconfined compression tests on the samples.

The goal for the shallow foundation elements is to create a solidified mass of sandy soil below the footprint of the footing as a minimum. The goal for the deep foundation element is to create a zone of solidified sand around it.

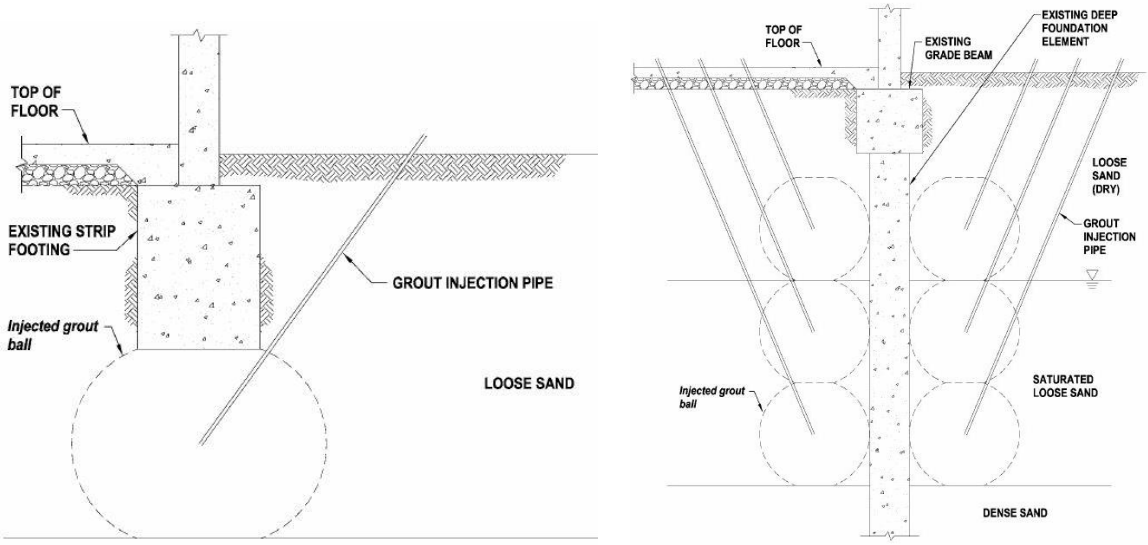


Figure 11.6. Permeation grouting under existing shallow foundation (left) and deep foundation (right) [1].

Cost/Disruption

Permeation grouting can be quite disruptive and costly especially if injection holes have to be drilled through existing concrete footings and slabs. If current operations in the building are to continue, the usual approach is to do the permeation grouting at night and clean up the work area before the start of work the next morning.

In the case of grouting under shallow foundations, there is a tendency for grout to migrate down, resulting in a weakly cemented lens of sand immediately below the shallow foundation elements. The tendency can be minimized by ensuring that grouting is performed in a strictly primary-secondary sequence.

12 ANNEX 2

Two important rehabilitation techniques, seismic isolation and damping, can be applied to any building type, and cannot be described as a local techniques to a specific structure members. Moreover, due to the fact that these measure are very advanced and require specific and deep knowledge, they are described independently in the following annex.

12.1 Seismic Isolation

To reduce the potential damage caused by earthquakes, past methods increased the building rigidity by adding shear walls or braced frames. The seismic Base isolation system is a flexible approach for isolating the structure from the ground, reducing seismic shock propagation into the structure.
The base isolation involves lengthening a building's fundamental period of vibration to reduce the seismic demand transmitted from the ground to the building.
In addition to reducing the chance of structural damage, the system also minimizes secondary damage to equipment inside the building such as computers, precision instruments, medical equipment and communications systems.

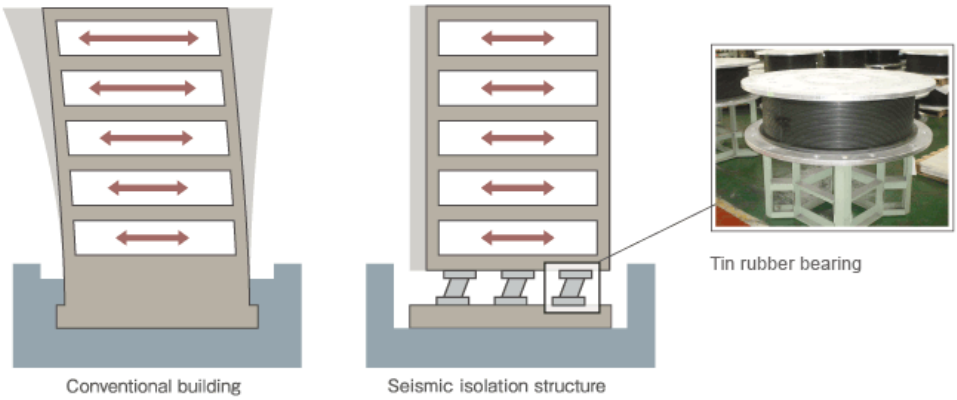


Figure 12.1. Seismic isolation scheme.

Figure 12.2. Seismic isolation example.

Types of isolation components: isolation components include elastomeric bearings and sliding bearings.

Elastomeric isolator consists of steel and rubber; it made of sandwiches of soft rubber sheets and hard steel (Figure 12.2). It works as a bearing to sustain the weight of the building and is able to move the building laterally. Soft rubber reduces the building vibration to slow shaking, and hard steel plate contributes to sustain the weight of building.
The slider has a coating of PTFE (Polytetrafluoroethylene) and a stainless steel plate finished with smooth surface as a mirror (Figure 12.3). It works as bearing to sustain the weight of the building and is able to move the building laterally on the surface of the plate with a certain amount of friction.

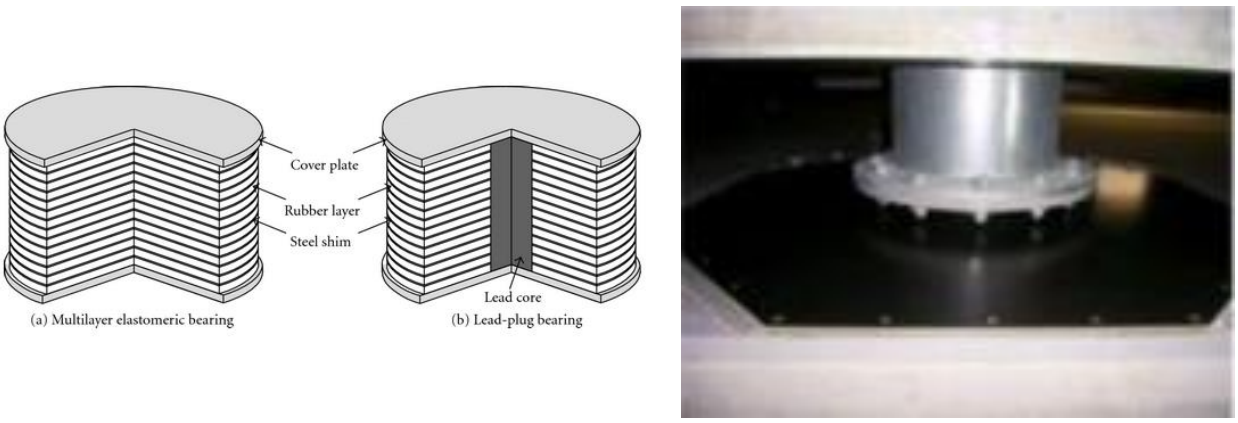


Figure 12.3. Elastomeric isolator (left) and slider isolator (right).

Applicable buildings: The period range for isolated buildings is from about 2 seconds to 4 seconds. As such, buildings on very soft soils and very tall, flexible buildings may not achieve much benefit from isolation. Seismic isolation is usually a very expensive rehabilitation strategy.

12.2 Energy Dissipation

The main reason to use passive energy dissipation devices in a structure is to dissipate much of the earthquake-induced energy in disposable elements not forming part of the gravity framing system, and to limit damaging deformations in structural components. Adding damping to an existing structure is a relatively unusual seismic rehabilitation strategy.
Supplemental damping hardware is parsed into three categories: hysteric, velocity-dependent and others. Examples of hysteric (displacement-dependent) dampers (Figure 12.4) include devices that exhibit rigid-plastic (friction devices), bilinear (metallic yielding devices) or trilinear hysteresis.
Velocity-dependent devices (Figure 12.5) include solid and fluid viscoelastic devices and fluid viscous devices. There are other devices as well, including shape-memory alloys, friction-spring assemblies with re-centering capability, and fluid restoring force-damping devices [31].

Figure 12.4. Metallic yielding damper example.

Figure 12.5. Fluid damper example (Courtesy of Cameron Black, SIE Inc).

Applicable buildings: Most engineers believe that adding damping is most relevant in flexible buildings, such as steel or concrete moment frames. Damping is also a common element in the seismic isolation system, but there it must accommodate very large displacements.

Figure 12.6. Damper example.

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