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Guidelines for Retrofitting of Existing Buildings

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LIST OF ACRONYMS:

ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
BPO	Basic Performance Objective
CP	Collapse Prevention
DCR	Demand-capacity ratio
EPO	Enhanced Performance Objective
FEMA	Federal Emergency Management Agency
IO	Immediate Occupancy
κ	Knowledge Factor
LS	Life Safety
LSP	Linear Static Procedure
NDP	Nonlinear Dynamic Procedure
NSP	Nonlinear Static Procedure
Q_{CE}	The expected strength (capacity) of the component
Q_{UD}	The force due to the gravity and earthquake loads
S_1	The 5%-damped response spectrum ordinate for long periods (1.0 second)
S_s	The 5%- damped response spectrum ordinate for short periods (0.2 second)
T_R	Return Period

INTRODUCTION

This report provides summarized guidelines for the systematic assessment of the seismic performance of existing building structures and the appropriate rehabilitation (retrofitting) strategies that can be implemented to improve such a performance. The current report is mainly based on state-of-the-art codes, standards, and guidelines that have been developed in the US by various specialized institutions such as the Federal Emergency Management Agency (FEMA), the American Society of Civil Engineers (ASCE), and the American Concrete Institute (ACI). Examples for those codes/standards/guidelines include FEMA 356 (FEMA 2000), ASCE 41-13 (ASCE 2013), FEMA P-58 (Applied Technology Council 2018), ACI 440 (ACI 440.2R 2008) and FEMA 547 (FEMA 2006). This report is divided into two main parts:

Part 1: Assessment of existing buildings:

This part incorporates a systematic procedure for the assessment of existing building structures following the state-of-the-art concepts of performance-based earthquake engineering. Such procedures provide easy-to-measure performance metrics that represent a strong indicator for the capacity of the building, which can be compared with user-specific performance objectives and acceptance criteria. Different analysis and modeling techniques are also discussed in this part of the report, which vary with respect to complexity, accuracy, computational time, and effort (e.g., linear static, linear dynamic, nonlinear static, and nonlinear dynamic procedures). Each one of those techniques has its own limitations and conditions that must be achieved in order to guarantee accuracy of results.

Part 2: Rehabilitation (retrofitting) of existing buildings:

The second part of the current report incorporates a detailed summary for the rehabilitation (retrofitting) strategies that can be adopted to improve the seismic performance of existing structures that do not comply with the desired performance objectives identified in Part 1. The rehabilitation strategies in accordance with the type of structural deficiency (e.g., global strength, global stiffness, ductility, (local) component detailing, diaphragms, foundations, and architectural configuration). They also vary with respect to the required type of intervention (e.g., adding new elements, enhance the performance of existing elements, reduce seismic demands, remove elements, improve the connection between structural components). The proposed rehabilitation strategies depend on the type of structure under consideration, and availability of specialized labor, equipment, and materials. The rehabilitation strategies are consistent with the characteristics of Palestinian buildings and Palestinian engineering practice.

1. ASSESSMENT OF EXISTING BUILDINGS:

The requirements and provisions for the assessment of existing building structures in the current document are based on the performance-based earthquake engineering approach. Those differ substantially from the traditional seismic design procedures for new buildings that can be found in the available building codes and/or standards. Traditional design procedures of new buildings assume an elastic behavior of structures during the modeling phase. While this assumption simplifies the work of engineers, it does not convey the actual performance of the building as large nonlinear deformations are expected to develop against strong earthquake-induced ground shaking. Such assumption could be useful in the case of new buildings, whereas it becomes problematic in the case of existing structures due to the high uncertainties pertaining to the prediction and simulation of their performance. This chapter provides a brief summary for a systematic procedure that can be implemented for the assessment of existing buildings, which vary in terms of analysis, required expertise level, and input information.

1.1. Performance Objectives and Seismic Hazard

1.1.1. Definition of performance objectives and seismic damage levels

The seismic performance of existing structures is measured based on pre-selected performance objectives. The performance objective represents satisfying a specific structural damage (performance) level under an appropriate seismic hazard level. If the existing structure does not meet the intended performance objective, then retrofitting (rehabilitation) is needed. Two important performance objectives are discussed in Table 1.1.

Table 1.1 Performance objectives to measure seismic performance

Hazard level	Seismic Performance Objective	
	Basic Performance Objective	Enhanced Performance Objective
475 years	Life safety (Significant Damage)	-
2475 years	Collapse Prevention (Near Collapse)	Life safety (Significant Damage)

As seen in Table 1.1, the seismic hazard levels are expressed in terms of return period (T_R). Two hazard levels can be found: 1) 475 years equivalent to 10% exceedance probability in 50 years; 2) 2475 years corresponding to 2% exceedance probability in 50 years. To achieve the Basic Performance Objective (BPO), the structure must not exceed the life safety (LS) level of performance (i.e., significant damage) -or better- against seismic loads corresponding to T_R equal to 475 years, and it should achieve collapse prevention (CP) -or better- at T_R of 2475 years. On the contrary, achieving the Enhanced Performance Objective (EPO) requires achieving the LS level of performance or better (e.g., immediate occupancy – IO) against seismic loads corresponding to T_R equal to 2475 years. Additional description of both the LS and CP levels is provided in Table 1.2.

Table 1.2 Description of life safety and near collapse levels

Performance (damage) level	Generic description	Damage observations in reinforced-concrete moment-resisting frames
Life safety (LS)	Significant damage to the structure, but some margin against partial or total structural collapse remains	Extensive beam damage. Spalling of cover and shear cracking (<1/8" width) for ductile columns. Minor spalling in nonductile columns and joint cracks <1/8" wide
Collapse prevention (CP)	Building is on the collapse verge with extensive damage as well as significant degradation in stiffness and strength. The structure remains able to carry gravity loads	Extensive cracking and formation of hinges in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns

Structures assessed and retrofitted to meet the BPO are expected to experience little damage from frequent and moderate earthquake events, but significantly more severe damage from strong and infrequent earthquakes. The level of damage and economic losses experienced by such buildings might be greater than those sustained by new buildings. Accordingly, the PBO can be used as a target for retrofitting design for ordinary buildings (e.g., residential, offices). On the contrary, structures retrofitted to meet the EPO are expected to perform better and experience less damage compared to those meeting the BPO. Therefore, the EPO is more applicable to critical buildings, where it is required to maintain functionality, both during and after earthquake-induced ground shaking.

1.1.2. Definition of seismic hazard and response spectrum

The seismic hazard associated with ground shaking must be defined for the site of interest in accordance with local building codes/standards and hazard maps. The seismic hazard generally can be represented by either developing the acceleration response spectrum for the site under consideration or selecting hazard-consistent ground-motion records. The latter choice is only needed if it is intended to perform time-history analysis as explained later.

<u>Acceleration response spectrum</u>:

To be able to construct the acceleration response spectrum, two main parameters are needed, which can be obtained from seismic hazard maps specifically-developed for the area under consideration. Those two parameters are defined as follows:

1. The 5%- damped response spectrum ordinate for short periods (0.2 second) known as S_S.

2. The 5%-damped response spectrum ordinate for long periods (1.0 second) known as S_1.

If it is required to establish an acceleration response spectrum corresponding to T_R equal to 475 years, then the hazard maps used to obtain both S_s and S_1 must be consistent with such a hazard level. The same also applies if it is needed to develop the same spectrum but considering T_R of 2475 years. It should be noted that if maps of S_s and S_1 for T_R equal to 475 years, it is possible to use S_s and S_1 derived from the 2475-year maps, multiplied by 2/3. Prior to evaluating the final acceleration response spectrum, it is required to modify both S_s and S_1 according to the site soil class to account for the possibility of seismic-wave amplification, particularly in soft soil areas. This modification is achieved using Eqs. (1.1) and (1.2).

$$S_{xs} = F_a S_s \tag{1.1}$$

$$S_{x1} = F_v S_1 \tag{1.2}$$

where F_a and F_v are modification factors that can be determined from Table 1.3 and Table 1.4, respectively. Soil classes are defined by letters ranging from A (hard rock) to E (soft clay). If the site soil is vulnerable to potential failure/collapse (e.g., peats or highly-organic clays), then the classification becomes "F". In such a case, a site-specific geotechnical evaluation must be conducted to determine the soil coefficients.

Table 1.3 Values of factor F_a to account for site class effects on S_s

Soil class	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9

Table 1.4 Values of factor F_v to account for site class effects on S_1

Soil class	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.4$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4

1.2. Obtain As-built Information

The first step in the assessment process constitutes obtaining the as-built information related to the existing building including the detailing, type, material characteristics, and condition of the structural components. Information must be also obtained regarding non-structural elements that contribute to the overall performance of the building such as masonry infill walls. Such data can be acquired from available structural/architectural drawings and specifications. This information shall be also supported by on-site investigations that might incorporate visual inspections, testing of materials, foundation assessment, and nondestructive examination of building components. At least one site visit must be carried out to assure that the gathered as-built information represents the existing conditions. It is also possible to conduct interviews with building owners, tenants, or the original architect/contractor. More important remarks on the supply of existing building information are reported in the following sub-sections.

1.2.1. Building configuration information

The as-built configuration information must include the type/arrangement of existing structural elements of the gravity and lateral-load-resisting systems. It shall also include the nonstructural components that affect either the stiffness and/or strength of the structural elements.

1.2.2. Properties of components

To facilitate performing structural analysis to assess the performance of existing buildings, it is required to obtain information on the properties of existing components (e.g., beams, columns, diaphragms) in addition to material strength characteristics. This in turn permits the calculation of strength and deformation capacity of the structural elements. It should be noted that a knowledge factor κ must be used in conjunction with material properties during the evaluation of capacities of building components to account for their uncertainty level. This factor reflects the confidence level with which the properties of building components are known. The value of κ can be selected depending on the level of knowledge and testing conditions in accordance with Table 1.5.

Table 1.5 Values of knowledge factor (κ)

Level of knowledge	Minimum	Usual	Comprehensive
Testing conditions	No tests	Usual testing	Comprehensive testing
Material properties	From default values or drawings	From drawings and tests	From drawings and tests
Knowledge factor (κ)	0.75	1.00	1.00

It should be noted that the κ values in Table 1.5 are proposed assuming the existing structure will be retrofitted to satisfy the basic performance objective (BPO). If the level of knowledge is classified as usual, and the retrofitting will target a higher performance objective than the PBOE (e.g., enhanced performance objective), then κ must be reduced to 0.75.

1.2.3. Adjacent buildings

Sufficient information shall be gathered on the configuration of adjacent structures to account for any potential interaction between those and the structure under assessment. Such information is needed only if adjacent structures are located within 4% of the height above grade at the location of impact. Information must be also collected on any shared structural elements between adjacent buildings such as shear walls. The gathered information shall also address the possibility of hazards from adjacent buildings, particularly falling debris, fire, explosion, or chemical leakage.

1.2.4. Data collection requirements

The data collected on the as-built structure can be classified with respect to the level of knowledge as minimum, usual, or comprehensive. Those are explained in the below points:

Minimum knowledge level:

- Information is obtained from design drawings with sufficient information to analyze component demands and capacities. Design drawings need not be complete but must at least communicate the configuration of the gravity and lateral-force-resisting systems with sufficient level of detail to carry out linear elastic analysis.
- If information from design drawings is unavailable, incomplete, or non-existent, it shall be supplemented by a comprehensive condition assessment that incorporates destructive and nondestructive testing/investigation.
- If material test records are absent, the default material properties shall be used based on the construction age and applicable guidelines/standards.
- Data on adjacent buildings must be gathered from field surveys and any available as-built information.
- Information on site and foundation shall be collected as well.

Usual knowledge level:

- Information is obtained from design drawings with sufficient information to analyze component demands and capacities. Design drawings need not be complete but must at least communicate the configuration of the gravity and lateral-force-resisting systems with sufficient level of detail to carry out any type of analysis procedure.

- If information from design drawings is unavailable, incomplete, or non-existent, it shall be supplemented by a comprehensive condition assessment that incorporates destructive and nondestructive testing/investigation.
- If material test records are absent, material properties shall be determined by usual material testing.
- Data on adjacent buildings must be gathered from field surveys and any available as-built information.
- Information on site and foundation shall be collected as well.

Comprehensive knowledge level:

- Data is obtained from construction documents including design drawings, specifications, material test records, quality assurance reports that cover original construction and any subsequent modifications. Such information must be verified through visual assessment.
- If construction documents are incomplete, missing information shall be supplemented by a comprehensive condition assessment incorporating destructive and nondestructive testing/ investigation.
- If material test records are absent, material properties shall be determined by comprehensive material testing. The coefficient of variation in such tests must be less than 20%.
- Data on adjacent buildings must be gathered from field surveys and any available as-built information.
- Information on site and foundation shall be collected as well.

Finally, it should be noted that if an existing building is to be assessed using linear elastic analysis procedures, the as-built data in such a case can be collected in accordance with the minimum level of knowledge. On the other hand, nonlinear analysis procedures require either the usual or comprehensive levels of knowledge.

1.3. Analysis Procedures & Modelling Assumptions: Generic Description

Structural analyses are needed to evaluate the force and deformation demands generated in different building components by ground shaking corresponding to a selected earthquake hazard level. Four analysis procedure are commonly used for seismic analysis: two linear procedures termed Linear Static Procedure (LSP) and Linear Dynamic Procedure (LDP); two nonlinear procedures known as Nonlinear Static Procedure (NSP) and Nonlinear Dynamic Procedure (NDP). The linear procedures (LSP and LDP) presented in this document for the assessment of existing buildings maintain the traditional assumption of a linear elastic stress-strain relationship, but they introduce some adjustments to the overall building deformations to allow a better characterization of the actual nonlinear seismic response. The nonlinear procedures (NSP and NDP), conversely, adopt the actual nonlinear stress-strain relationship to facilitate simulating the actual complex nonlinear behavior of the building. Those procedures, however, require considerably more experience and judgement to be performed compared to the linear procedures. More details are described in the below sub-sections.

1.3.1. Linear procedures

Linear procedures are only applicable to buildings with no irregularities unless the building is capable of responding to seismic excitations in a "nearly" linear elastic manner. If the building is irregular, some checks must be performed to confirm whether it is permitted to proceed with linear procedures or not. To do so, the magnitude and distribution of inelastic demands are approximated by calculating the demand-capacity ratio (DCR) for each primary structural element in accordance with Eq. (1.3)

$$DCR = \frac{Q_{UD}}{Q_{CE}} \tag{1.3}$$

where Q_{UD} is the force due to the gravity and earthquake loads and Q_{CE} is the expected strength (capacity) of the component. The DCRs must be calculated for each type of internal force (e.g., axial, bending moment, shear) of each primary structural component. The critical loading case for each element is the one with the largest DCR value. The applicability of linear procedures is decided as per the following points:

1. If the DCR values for all components are ≤ 2.0, then linear procedures are applicable.
2. If one or more component DCR values exceed 2.0, and no irregularities are found, then linear procedures are still applicable.
3. If one or more component DCR values exceed 2.0, and the structure has irregularities, then linear procedures are not applicable, and are not permitted to be used.

The irregularities mentioned previously incorporate the in-plane discontinuity, out-of-plane discontinuity, severe weak (soft) story, and severe irregularity. Those are schematically illustrated in Figure 1.1. It should be noted that the severe weak story is assumed to exist in any direction of the building if the ratio of the average DCR to that of an adjacent story exceeds 125%. The severe torsional irregularity is considered to exist if the ratio of the critical element DCR values on one side of the center of resistance of a story, to those on the other side of the center of resistance, exceeds 1.5.

It is important to understand that in the case of reinforced-concrete members analyzed using linear procedures, it is essential to model the effective stiffness that takes into account the initial cracking rather than using the gross stiffness value. For simplicity, this can be addressed by reducing the rigidity of different members following the rules established in Table 1.6. E_c is the concrete modulus of elasticity and I_g is the gross second moment of inertia. A_w represents the web area, whilst A_g is the gross cross-sectional area. E_s and A_s are respectively the modulus elasticity of steel reinforcement and total area of longitudinal reinforcement.

Table 1.6 Effective stiffness for reinforced-concrete members assessed via linear procedures

Member (component)	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Non-prestressed beams	$0.5E_cI_g$	$0.4E_cA_w$	--
Prestressed beams	E_cI_g	$0.4E_cA_w$	--
Columns with compression due to design gravity loads $\geq 0.5A_gf_c'$	$0.7E_cI_g$	$0.4E_cA_w$	E_cA_g
Columns with compression due to design gravity loads $\leq 0.3A_gf_c'$	$0.5E_cI_g$	$0.4E_cA_w$	E_sA_s
Uncracked walls	$0.8E_cI_g$	$0.4E_cA_w$	E_cA_g
Cracked walls	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g

1.3.2. Nonlinear procedures

Nonlinear procedures are applicable to any building type regardless of the type retrofit strategy. They shall be also used in the cases where linear procedures are not permitted. The NSP for instance is a very reliable approach to capture the nonlinear behavior of structures compared to linear procedures. However, it is not exact and cannot account for higher mode effects. Therefore, the NSP is permitted in structures with negligible higher mode effects. To verify that, modal response spectrum analysis must be performed with modes sufficient to capture 90% of the total mass. A second modal response spectrum analysis must be run considering only the first mode participation. The higher mode effects are considered significant if the shear in any story resulting from the former case exceeds 130% of the corresponding story shear evaluated using the latter case. If higher-mode effects are significant, it is still permitted to use NSP, provided that additional LDP analysis is also performed to supplement the NSP. Otherwise, only the NDP can be implemented. The results of NDP shall be reviewed by a third-party engineer with experience in seismic design and nonlinear procedures.

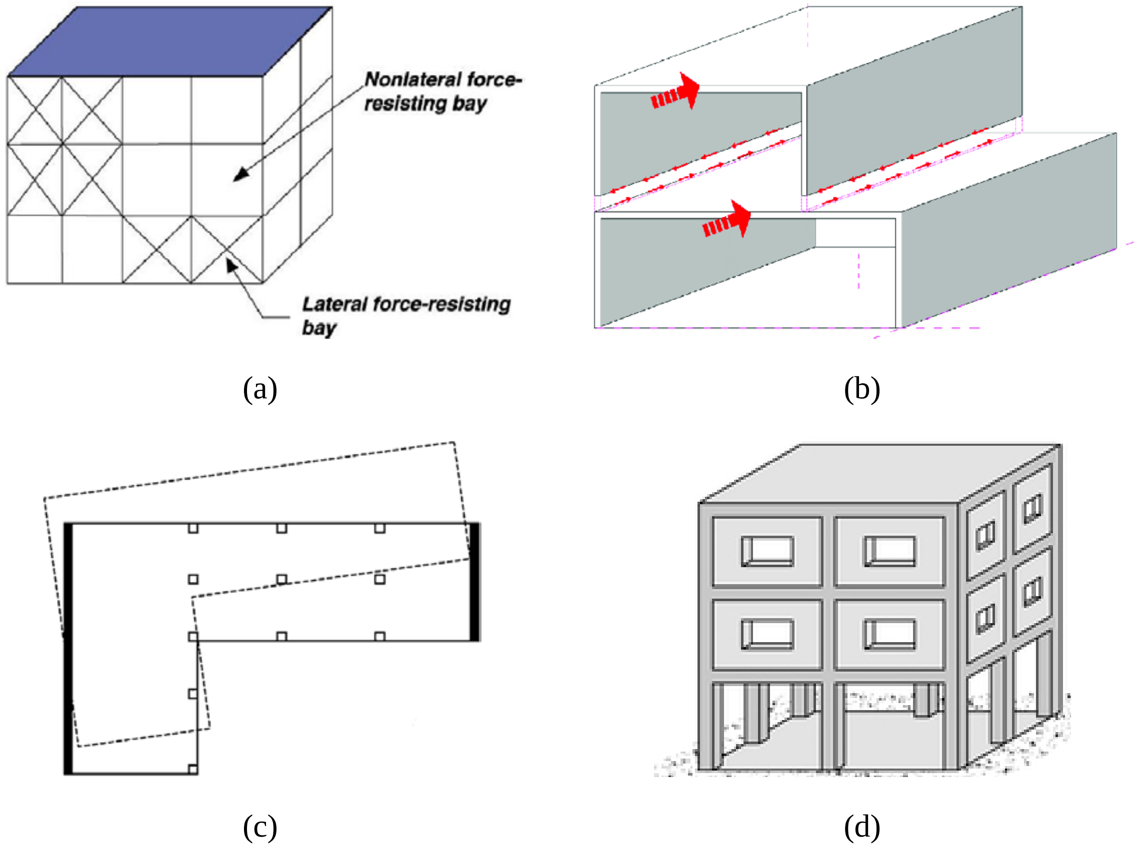


Figure 1.1 Common structural irregularities including: a) in-plane discontinuity; b) out-of-plane discontinuity; c) severe torsional irregularity; and 4) severe weak (soft) story irregularity.

1.3.3. Basic modelling assumptions

Ideally, three-dimensional numerical models must be used to analyze and evaluate the seismic performance of existing structures. The use of two-dimensional models is permitted, provided that the building under investigation meets one the following conditions:

1. The diaphragms are rigid
2. Torsional effects are not very significant.
3. If the building has a flexible diaphragm and/or torsional exceeds the appropriate limits, two-dimensional models can be adopted provided the structure meets extra requirements related to modelling and demand calculations. Those can be found in FEMA 356.

It should be noted that the P-Δ effects shall be also considered in the adopted numerical modelling both in two- and three-dimensional models.

1.3.4. Combination of gravity and seismic actions (loads)

The overall gravity loading, Q_G, shall be considered in combination with seismic loads in accordance with Eq. (1.4):

$$Q_G = 1.1(Q_D + Q_L + Q_S) \tag{1.4}$$

When the effects of gravity and seismic loads are counteracting, i.e., the resulting internal forces have different directions, the gravity loads must be evaluated as per Eq. (1.5):

$$Q_G = 0.9(Q_G) \tag{1.5}$$

where Q_D is the dead load, Q_L is the effective live load, equal to 25% of the unreduced live load used for the design, and Q_S is the effective snow load.

1.4. Implementation of Analysis Procedures

This section explains how the various analysis procedures, introduced previously, can be implemented to generate the different structural response parameters (e.g., component forces, deformations) from the numerical model developed to simulate the building behavior.

1.4.1. Linear static procedure (LSP)

If the LSP is selected for analyzing the building, the seismic forces, and their distribution over the building height, in addition to the corresponding internal forces and displacement demands shall be evaluated based on the requirements of this sub-section. Recall that buildings in the LSP must be subjected to a (pseudo-) lateral seismic load, in order to enable the calculation of such internal forces and displacement demands. It is important to know that the magnitude of the pseudo seismic load has been selected in a way that, when applied to linear elastic models, it will result in displacement values that approximate the maximum displacements expected during the design earthquake. The aforementioned pseudo seismic load in a given horizontal direction of a building can be calculated using Eq. (1.6):

$$V = C_1\, C_2\, C_3\, C_m\, S_a\, W \tag{1.6}$$

where V is the pseudo seismic load, C_1 is a modification factor to relate the expected maximum inelastic displacements to displacements calculated from linear elastic analysis. C_1 can be assumed 1.5 for fundamental vibration periods (T) < 0.10 second and shall be assumed equal to 1.0 for $T \geq$ the period at the end of the constant spectral acceleration plateau (T_S) in the response spectrum used to define the seismic demand. Linear interpolation can be used to calculate C_1 for any intermediate T value. C_2 is a modification factor accounting for the pinched hysteresis shape and stiffness degradation. It is taken equal to 1.0 for linear procedures.

Moreover, C_3 is a modification factor to address the increased displacement demands due to dynamic P-Δ effects. C_3 depends on the value of the stability coefficient θ_i calculated as per Eq. (1.7). If θ_i is less than 0.1 in all stories, C_3 can be assumed 1.0, otherwise, it is taken as

$1 + 5(\theta - 0.1)/T$ using θ equal to the maximum θ_i value among all stories. Finally, C_m is the effective mass factor to account for the mass participation of higher modes. It can be taken as 1.0 if T is greater than 1.0 second, otherwise it must be selected according to Table 1.7. C_m is the response spectrum acceleration at the T and damping ratio of the building. W is the effective seismic weight of the building.

Table 1.7 Values of effective mass factor C_m

Number of stories	Concrete moment frames	Concrete shear walls	Steel moment frame	Steel concentric braced frame
1 – 2	1.0	1.0	1.0	1.0
3 or more	0.9	0.8	0.9	0.9

$$\theta_i = \frac{P_i \, \delta_i}{V_i \, h_i} \tag{1.7}$$

where P_i is the portion of the total weight of the structure, including dead, permanent live load, and 25% of transient live loads acting at story i. V_i is the seismic shear force acting on story i. h_i is the height of story i, whereas δ_i is the lateral drift of story i at its center of rigidity.

The calculated pseudo seismic force must be distributed vertically across different stories. The lateral force applied at any story level x, F_x shall be computed as per Eq. (1.8).

$$F_x = \frac{w_x \, h_x^k}{\sum_{i=1}^{n} w_i \, h_i^k} \cdot V \tag{1.8}$$

where k is 2.5 for $T \geq 2.5$ seconds and 1.0 for $T \leq 0.5$ seconds (linear interpolation can be used for any intermediate values). w_i is the portion of total building weight W located on floor level i. w_x is the portion of total building weight W located on floor level x. h_i is the height from the base to floor level i, and h_x is the height from the base to floor level x.

1.4.2. Linear dynamic procedure (LDP)

The LDP utilizes linear elastic numerical models, similar to those used for the LSP. The ground motions to be used in LDP can be characterized using one of the following:

1. A response spectrum to be used in conducting modal response spectrum analysis. A sufficient number of modes must be selected to at least capture 90% of the total seismic mass. Peak member forces and displacements shall be combined either using the square root sum of squares (SRSS) or the complete quadratic combination (CQC) rule.

2. Ground-motion acceleration records to be used in time-history analysis. If three or more ground-motion records are used, then the maximum response (e.g., force, deformation) must be finally taken. If, however, seven or more consistent pairs of horizontal ground-motion records are implemented, the average response resulting from all of them can be adopted.

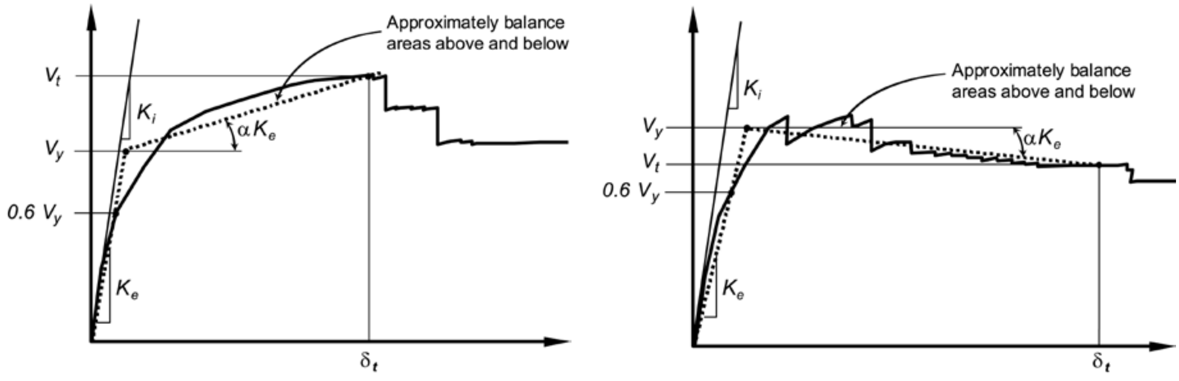
It should be noted that all the forces and deformations calculated using either the modal response spectrum analysis or time history analysis methods shall be multiplied by the modification factors C_1, C_2, and C_3 defined in section 1.4.1.

1.4.3. Nonlinear static procedure (NSP)

When using the NSP, the numerical model of the building shall explicitly incorporate the nonlinear load-deformation characteristics for all components. Those are called backbone curves, and they shall address strength degradation and residual strength, if any. Buildings in NSP are subjected to monotonically increasing lateral loads until reaching a specific target displacement. Such a value represents the maximum displacement likely to be experienced by the structure during the design earthquake. NSP is also known as "pushover" analysis.

The control node at which the displacement response of the structure is monitored must be located at the center of mass at the top floor of the building. The lateral load shall be applied using two different patterns. The first one is a modal pattern that follows the relative floor displacements associated with the first vibration mode, whereas the second is a uniform pattern that consists of loads equally distributed across different floor levels.

During performing the NSP (pushover analysis), the relationship between the base shear and the displacement (pushover curve) at the control node is recorded. It shall be then replaced with an equivalent idealized to enable estimating the effective lateral stiffness (K_e), effective yield strength (V_y), as illustrated in Figure 1.2. This relationship should be bilinear with initial slope equal to K_e and post-yield slope of αK_e. The line segments shall be located through utilizing an iterative procedure that approximately balances the area above and below the curve. K_e is taken as the secant stiffness corresponding to a base shear equal to 60% of V_y. The post-yield slope (αK_e) shall be determined by a line that passes through the actual pushover curve at the target displacement (δ_t) as clarified in Figure 1.2.



(a) (b)

Figure 1.2 Idealized force-displacement (pushover) curves for: a) positive post-yield slope; and (b) negative post-yield slope, after FEMA 356 (FEMA 2000).

The target displacement, δ_t, must be evaluated at each floor level in accordance with Eq. (1.9). The effective period of vibration (T_e) is quantified using Eq. (1.10), and it is a function of T, K_e, and the elastic lateral stiffness (K_i).

$$\delta_t = C_0\ C_1\ C_2\ C_3\ S_a \frac{T_e^2}{4\pi^2} g \tag{1.9}$$

$$T_e = T\sqrt{\frac{K_i}{K_e}} \tag{1.10}$$

where C_0 represents a modification factor to relate the spectral displacement of an equivalent single-degree-of-freedom (SDoF) system to the top-floor displacement of the multi-degree-of-freedom (MDoF) system. C_0 can be selected from Table 1.8. C_1 relates the expected maximum inelastic displacement to displacements calculated from linear elastic response. It can be taken as 1.0 for $T_e \geq T_S$ and as $[1.0 + (R-1)T_e/T_S]/R$ for $T_e < T_S$, R is the ratio of elastic strength demand to yield strength calculated as per Eq. (1.11). C_2 is a modification factor to account for the effect of pinched hysteretic shape and stiffness degradation. A value of 1.0 can be assumed. C_3 is a modification factor to address the increased displacements due to P-Δ effects. For buildings with positive post-yield stiffness, it can be taken as 1.0. For buildings with negative post-yield stiffness, C_3 shall be estimated using Eq. (1.12). g is the acceleration of gravity. It should be noted that C_m is defined in section 1.4.1.

$$R = \frac{S_a}{V_y/W} \cdot C_m \tag{1.11}$$

$$C_3 = 1.0 + \frac{|\alpha|(R-1)^{1.5}}{T_e} \tag{1.12}$$

Table 1.8 Values of the modification factor C_0[1]

Type	Shear buildings		Other buildings
Number of stories	Modal load pattern	Uniform load pattern	Any load pattern
1	1.00	1.00	1.00
2	1.20	1.15	1.20
3	1.20	1.20	1.30
5	1.30	1.20	1.40
10+	1.30	1.20	1.50

[1]Linear interpolation shall be used for intermediate values.

1.4.4. Nonlinear dynamic procedure (NDP)

When selecting the NDP for seismic analysis, the building numerical model must directly incorporate the nonlinear load-deformation characteristics of each individual component. Modelling assumptions related to the NDP are similar to those related to the NSP, except for the control node and target displacement. Ground-motion records must be selected to perform non-linear time-history analysis.

Ground-motion records shall not be less than three sets, each one containing two horizontal components. The selected ground-motion records shall have magnitudes, source distances, and fault mechanisms that are similar to the design earthquake in the site under consideration. Records can be also scaled or spectrally matched with a target response spectrum to ensure hazard consistency. If three record sets are used in analyzing the structure, the maximum value of each response parameter (e.g., member force, displacement at a specific floor level) shall be used for the final assessment. Where seven or more record sets are deployed, the average value of each response parameter can be evaluated instead of the maximum.

1.5. Acceptance Criteria and Numerical Modeling Parameters

Upon implementing any of the analysis procedures explained in section 1.4 to assess existing buildings, the acceptability of the resulting force and deformation in structural members shall be compared with the so-called “acceptance criteria”. However, structural members must be first divided into two sub-categories as follows:

1. Primary members: this term refers to any structural member that contributes to collapse resistance during an earthquake-induced ground-shaking event.
2. Secondary members: all remaining members.

It should be understood that in a typical building, nearly all structural members contribute to its stiffness, damping, and mass during ground shaking. However, not all those members can provide a significant contribution for collapse resistance. For simplicity, any member that does not contribute to earthquake resistance significantly due to its low stiffness or strength might be designated as a secondary member. Generally speaking, the use of secondary classification for some members will allow them to experience greater damage and nonlinear deformation compared to primary members.

Each action acting on a structural member must be also classified as deformation-controlled (ductile) and force-controlled (non-ductile). Figure 1.3 illustrates different types of force-deformation ($Q-\Delta$) relationships.

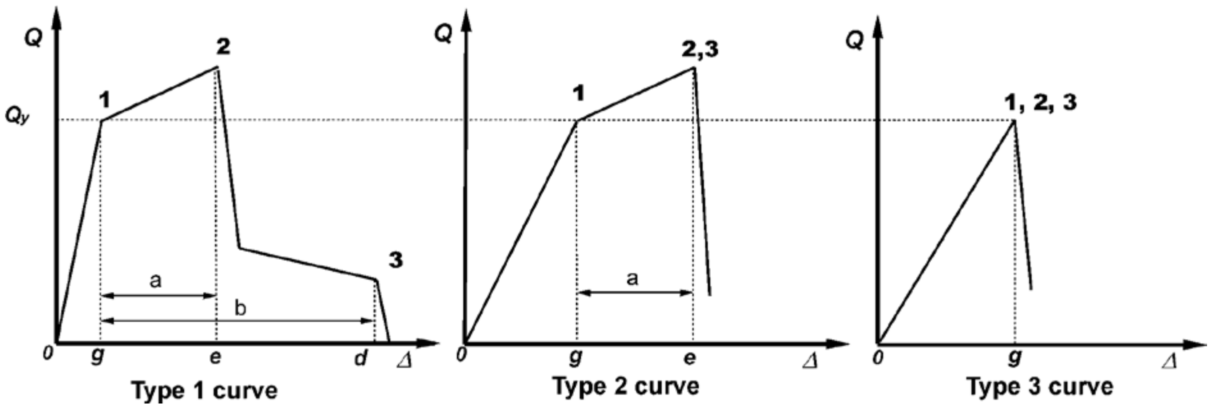


Figure 1.3 Different component force-deformation ($Q-\Delta$) relationships, after FEMA 356.

Type 1 curve shown in Figure 1.3 represents a ductile behavior, where an elastic response initiates from point 0 to 1, followed by a plastic range that constitutes hardening from point 1 to 2 and softening from point 2 to 3. The residual strength here would be sufficient to at least sustain gravity loads. Type 2 curve accounts for ductile behavior characterized by a loss of strength and ability to support gravity actions upon the end of the softening branch. On the other hand, Type 3 curve depicts a brittle non-ductile behavior where there is a sudden loss of strength and ability to support gravity actions upon the end of the elastic branch. For simplicity, different actions may be classified as force- or deformation-controlled as per Table 1.9.

Table 1.9 Classification of deformation- and deformation- controlled actions

Component	Deformation-controlled action	Force-controlled action
Moment frames:		
• Beams	• Moment (M)	• Shear (V)
• Columns	• M	• Axial load (P) and V
• Joints	• --	• V
Shear walls	M, V	P
Braced frames:		
• Braces	• P	• --
• Beams	• --	• P
• Columns	• --	• P
• Shear links	• V	• P, M

In the current guidelines, a generalized force-deformation relationship is used to identify the modeling parameters of the component in addition to the acceptance criteria. This relationship is reported in Figure 1.4, where the y-axis shows the normalized force Q/Q_y (i.e., force divided by its yield value), and the x-axis addresses the deformation (displacement Δ or rotation θ). a and b refers to portions of deformation occurring upon the elastic branch (plastic). c indicates the reduced strength that takes place after the softening branch.

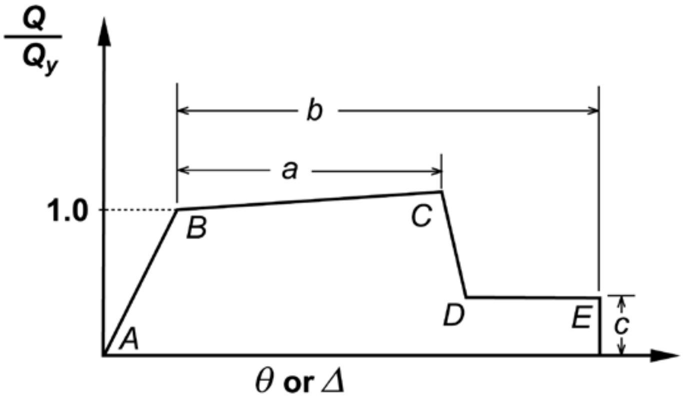


Figure 1.4 Generalized component force-deformation curves to depict modeling parameters, after FEMA 356 (FEMA 2000).

1.5.1. Numerical modeling parameters and acceptance criteria for linear procedures

The deformation-controlled design actions used in the linear procedures shall be formulated as per Eq. (1.13), where Q_{UD} is the deformation-controlled design action resulting from gravity and earthquake loading combined, Q_E is the action from earthquake forces, and Q_G is the action resulting from gravity loading:

$$Q_{UD} = Q_E \pm Q_G \tag{1.13}$$

The deformation-controlled design action (Q_{UD}) in primary and secondary structural members must satisfy Eq. (1.14):

$$m\kappa Q_{CE} \geq Q_{UD} \tag{1.14}$$

where κ is the knowledge factor defined in section 1.2.2, m is a component demand modifier (factor) to address the expected ductility associated with the action under consideration at the selected structural performance level. Finally, Q_{CE} is the expected strength of the component at the deformation level under consideration. To evaluate Q_{CE}, the mean values of tested material properties shall be adopted. It is permitted to use the standard procedures found in ACI 318 (ACI Committee 318 2014), but without using the strength reduction factors ($\emptyset$).

On the other hand, the force-controlled design actions (Q_{UF}) can be evaluated as follows:

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1\, C_2\, C_3\, J} \tag{1.15}$$

J is a force-delivery reduction factor, which can be taken as 2.0 in zones of high seismicity, 1.5 in zones of moderate seismicity, and 1.0 for low-seismicity zones. C_1, C_2, and C_3 have been already introduced in Eq. (1.6). The force-controlled design actions (Q_{UF}) in primary and secondary structural elements must satisfy Eq. (1.16):

$$\kappa Q_{CL} \geq Q_{UF} \tag{1.16}$$

where Q_{CL} is the lower-bound strength evaluated using the mean values of tested material properties minus one standard deviation. To evaluate Q_{CL}, it is permitted to use the standard procedures found in ACI 318 (ACI Committee 318 2014), but without using the strength reduction factors ($\emptyset$).For reinforced-concrete members, a factor of 1.50 can be used to convert lower-bound concrete compressive strength to an average (expected) value, whilst a factor of 1.25 can be used for reinforcing steel. Based on the previous discussion, Table 1.10 summarizes the numerical modeling parameters and acceptance criteria for the flexural behavior of reinforced-concrete beams assessed via linear procedures. The acceptance criteria are presented in the form of m-factors to be used in conjunction with Eq. (1.14). Table 1.10 also considers the member classification as either primary or secondary, in addition to the performance level under consideration (e.g., IO, LS, and CP).

Table 1.11 provides the same information, but for the reinforced-concrete columns, rather than beams. ρ and ρ' represent the tensile and compressive reinforcement ratios, respectively. ρ_{bal} is the reinforcement ratio producing balanced strain conditions. V accounts for the design shear force. b_w is the web width b, whereas d is the effective depth (i.e., distance from extreme compression fiber to the center of tensile reinforcement). f_c' is the compressive strength of concrete. Finally, P is the axial load acting on the column under consideration and P_o is the axial strength at zero eccentricity. A_g is the gross cross-sectional area.

It should be noted that acceptance criteria are also available in FEMA 356 for other reinforced-concrete structural members such as beam-column joints and shear walls. Other acceptance criteria are also available for different structural typologies such as steel and masonry structures. However, the current guidelines shed light specifically on reinforced-concrete frame members as they represent the dominant form of construction in Palestine.

Table 1.10 Modeling parameters and acceptance criteria for reinforced-concrete beams assessed using linear analysis procedures, after FEMA 356 (FEMA 2000).

Conditions			*m*-factors[3]				
			Performance Level				
				Component Type			
				Primary		Secondary	
			IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]							
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$					
≤ 0.0	C	≤ 3	3	6	7	6	10
≤ 0.0	C	≥ 6	2	3	4	3	5
≥ 0.5	C	≤ 3	2	3	4	3	5
≥ 0.5	C	≥ 6	2	2	3	2	4
≤ 0.0	NC	≤ 3	2	3	4	3	5
≤ 0.0	NC	≥ 6	1.25	2	3	2	4
≥ 0.5	NC	≤ 3	2	3	3	3	4
≥ 0.5	NC	≥ 6	1.25	2	2	2	3
ii. Beams controlled by shear[1]							
Stirrup spacing ≤ d/2			1.25	1.5	1.75	3	4
Stirrup spacing > d/2			1.25	1.5	1.75	2	3
iii. Beams controlled by inadequate development or splicing along the span[1]							
Stirrup spacing ≤ d/2			1.25	1.5	1.75	3	4
Stirrup spacing > d/2			1.25	1.5	1.75	2	3
iv. Beams controlled by inadequate embedment into beam-column joint[1]							
			2	2	3	3	4

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table shall be permitted.

Table 1.11 Modeling parameters and acceptance criteria for reinforced-concrete columns assessed using linear analysis procedures, after FEMA 356 (FEMA 2000).

Conditions			*m*-factors[4]				
			Performance Level				
				Component Type			
				Primary		Secondary	
			IO	LS	CP	LS	CP
i. Columns controlled by flexure[1]							
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$					
≤ 0.1	C	≤ 3	2	3	4	4	5
≤ 0.1	C	≥ 6	2	2.4	3.2	3.2	4
≥ 0.4	C	≤ 3	1.25	2	3	3	4
≥ 0.4	C	≥ 6	1.25	1.6	2.4	2.4	3.2
≤ 0.1	NC	≤ 3	2	2	3	2	3
≤ 0.1	NC	≥ 6	2	1.6	2.4	1.6	2.4
≥ 0.4	NC	≤ 3	1.25	1.5	2	1.5	2
≥ 0.4	NC	≥ 6	1.25	1.5	1.75	1	1.6
ii. Columns controlled by shear[1,3]							
Hoop spacing ≤ d/2, or $\frac{P}{A_g f'_c} \le 0.1$			–	–	–	2	3
Other cases			–	–	–	1.5	2
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]							
Hoop spacing ≤ d/2			1.25	1.5	1.75	3	4
Hoop spacing > d/2			–	–	–	2	3
iv. Columns with axial loads exceeding $0.70P_o$[1,3]							
Conforming hoops over the entire length			1	1	2	2	2
All other cases			–	–	–	1	1

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, columns must have transverse reinforcement consisting of hoops. Otherwise, actions shall be treated as force-controlled.
4. Linear interpolation between values listed in the table shall be permitted.

1.5.2. Numerical modeling parameters and acceptance criteria for non-linear procedures

The numerical modeling parameters and acceptance criteria for the flexural behavior of reinforced-concrete beams assessed via non-linear procedures are summarized in Table 1.12. Similar data are presented for reinforced-concrete columns in accordance with Table 1.13. It should be noted that the modeling parameters and acceptance criteria here are expressed in terms of plastic rotation angles (θ) in radians. The residual strength ratio (c) represents the ratio between the residual flexural strength and yield flexural strength.

Similarly, to the case of linear procedures, the acceptance criteria for reinforced-concrete members assessed through non-linear procedures differ with respect to the performance level (i.e., IO, LS, and CP) and whether the reinforced-concrete member (component) is classified as primary or secondary.

Table 1.12 Modeling parameters and acceptance criteria for reinforced-concrete beams assessed using non-linear analysis procedures, after FEMA 356 (FEMA 2000).

			Modeling Parameters[3]			Acceptance Criteria[3]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
Conditions			**a**	**b**	**c**	**IO**	**LS**	**CP**	**LS**	**CP**
i. Beams controlled by flexure[1]										
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.010	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.2	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.0020	0.0030	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.0	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.0020	0.0030	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]										
			0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table shall be permitted.

Table 1.13 Modeling parameters and acceptance criteria for reinforced-concrete columns assessed using non-linear analysis procedures, after FEMA 356 (FEMA 2000).

			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Performance Level				
			Plastic Rotation Angle, radians		Residual Strength Ratio		Component Type			
							Primary		Secondary	
Conditions			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008
ii. Columns controlled by shear[1, 3]										
All cases [5]			—	—	—	—	—	—	.0030	.0040
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]										
Hoop spacing ≤ d/2			0.01	0.02	0.4	0.005	0.005	0.01	0.01	0.02
Hoop spacing > d/2			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01
iv. Columns with axial loads exceeding $0.70P_o$[1, 3]										
Conforming hoops over the entire length			0.015	0.025	0.02	0.0	0.005	0.01	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, columns must have transverse reinforcement consisting of hoops. Otherwise, actions shall be treated as force-controlled.
4. Linear interpolation between values listed in the table shall be permitted.
5. For columns controlled by shear, see Section 6.5.2.4.2 for acceptance criteria.

2. REHABILITATION (RETROFITTING) OF EXISTING BUILDINGS:

2.1. General Introduction

The previous chapter explains performing detailed seismic assessment of an existing structure against selected performance objectives (e.g., BPO, EPO) and adopting different modeling and analysis procedures (e.g., linear, non-linear). In the case the structure of interest is found non-compliant with the suitable performance objective, an engineer can proceed with the design of a rehabilitation (retrofitting) strategy. Retrofitting strategies aim at modifying/improving key structural characteristics such as ductility, stiffness, and strength (see Figure 2.1), or at reducing earthquake-induced seismic demand. Several retrofitting techniques (systems) may be adopted to achieve one or more of the aforementioned strategies. For example, installing shear walls or lateral bracing can notably improve stiffness and lateral strength. Base isolation can reduce the seismic demands through decoupling the horizontal motion of the ground from the structure.

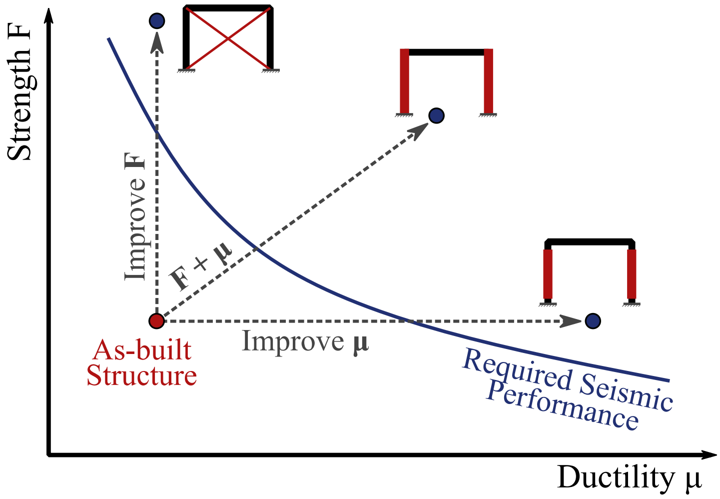


Figure 2.1 Effects of various retrofitting strategies on seismic performance, (Aljawhari et al. 2022).

This chapter provides a comprehensive illustration for the potential seismic deficiencies in Palestinian buildings, in addition to the most appropriate retrofitting strategies that are consistent with current construction practice and engineering reality.

2.1.1. Types of seismic deficiencies

There are several categories of seismic deficiencies that can affect the seismic performance of existing structures. A single structure might have one or more of such deficiencies. Those are classified in accordance with FEMA 547 (FEMA 2006) as follows:

1. Global strength:

This deficiency is quite common in older structures that lack seismic design, or they were designed using obsolete codes with inappropriate lateral strength requirements. Global strength can be defined as the lateral force at which the structure starts experiencing yielding at the global level, which can be identified through pushover analysis for example.

2. Global stiffness:

In most of the cases, strength and stiffness are provided by the same structural elements and retrofitting techniques. However, the two deficiencies (i.e., strength and stiffness) are often considered separately. Global stiffness refers to the overall stiffness of the entire lateral-force resisting system. A deficiency in the global system indicates that the structure is experiencing excessive drifts and deformation levels. A common example of such a deficiency is the severe drifts occurring on the first floor of a soft-story building.

3. Configuration:

This deficiency refers to configuration irregularities that can adversely influence the seismic performance of buildings. Such irregularities are classified into horizontal (plan), which can impose severe torsional stresses on structural elements for instance, and vertical irregularities that usually stem from uneven distribution of mass and stiffness along different floors.

4. Component detailing:

Detailing here indicates the design requirements that usually affect a component's or a structure's behavior in the non-linear range (beyond the traditional strength requirements). The most common example of such a deficiency is the poor confinement of reinforced-concrete beams and columns, which leads to lack of deformation ductility and faster degradation of lateral strength and stiffness.

5. Other deficiencies:

It is important to highlight that there are other deficiencies that might significantly affect the seismic performance of buildings under consideration. For instance, adjacent buildings can undergo pounding during seismic events due to the insufficient gap between them. Also, structural materials might be deteriorated or not adequate due to poor workmanship.

2.1.2. Classes of retrofitting (rehabilitation)

Retrofitting (rehabilitation) procedures can be divided into five different classes depending on the type of intervention to be made. Those are described in detail as follows:

1. Add new elements:

Adding new structural elements is among the most common classes of retrofitting. For instance, it is possible to add new shear walls, braced frames, or even moment frames to mitigate any deficiencies in global strength, stiffness, and configuration. New elements might be also added to account for any issues in the continuity of loading path. Construction wise, adding new elements to the existing buildings is one of the invasive solutions that might be sometimes very challenging to implement.

2. Enhance the performance of existing elements:

In this case, deficiencies can be addressed locally at the member (component) level. This is generally achieved through improving the flexural and/or shear strength of existing elements (mainly columns). It is also possible to improve the ductility of such individual elements in a way that allows them to experience higher non-linear deformation levels without collapsing or significant loss of lateral strength. For example, columns can be encased with jackets made of steel or reinforced concrete to provide additional shear and flexural strength as well as confinement. Wrapping columns with layers of fiber-reinforced polymers (FRP) has become a very popular option as well to enhance shear strength and confinement due to the ease of application and its very low invasiveness.

3. Improve connection between components:

This is needed in the case of weak beam-column connection that might affect the loading path for both gravity and earthquake-induced actions.

4. Reduce seismic demand:

For structures with a complete, but weak, lateral load-resisting system, removal of some top floors could be a very good solution to reduce seismic demand. This is usually adopted if there is excess space in the site to build new structures that accommodate the loss of the top floors from the existing building. The reduction of seismic demand can be also achieved through the alteration of the structure's dynamic response. The most common example of such a class is the seismic base isolation, which separates the horizontal motion of the ground from the motion of the structure itself. This retrofitting technique is relatively expensive compared to other traditional ones; therefore, it is usually applied for the preservation of historical buildings or for occupancies that cannot be disturbed.

Alternatively, it is possible to modify the dynamic response of the structure by using techniques that add more damping such as buckling-restrained bracing and tuned-mass dampers. Such techniques are economically more feasible than base isolation, and they help reduce the lateral deformation so that the structure becomes capable of experiencing acceptable damage levels due to earthquakes.

5. Remove selected components:

This can be performed by decoupling brittle structural elements from the lateral load-resisting system, or even removing them completely. For instance, it is very common to form slots between spandrel beams and columns to prevent the brittle “short” column failure that happens as a result of excessive shear stresses.

2.2. Common Palestinian Building Typologies

The current guidelines places emphasis on potential retrofitting measures that can applied to Palestinian buildings. Therefore, this section provides a detailed description of the most common building typologies that reflect the construction reality of Palestine. Those typologies are summarized as per the following:

1. Reinforced-concrete frame buildings (C1)
2. Reinforced-concrete frame buildings with soft story (C1a)
3. Shear-wall buildings (C2)
4. Unreinforced masonry buildings (URM)

More detailed description of those typologies and their performance is provided below.

2.2.1. Reinforced-concrete frame buildings (C1)

The C1 buildings constitute a reinforced-concrete framing system, in most of the cases composed of columns and beams. The lateral earthquake-induced forces here are sustained through moment frames that develop their overall stiffness from the rigid connections between columns and beams. The spaces between beams and columns in Palestinian frame buildings are commonly constructed with masonry infill walls made of cement blocks. Such walls can significantly improve the stiffness, lateral strength, and overall damping due to frame infill interaction. However, they might lead to stress concentration at column edges, thus leading to a significant increase in shear demand. This increase might lead to a brittle localized failure if the frame is not properly designed. Figure 2.2 provides an illustration of the C1 buildings.

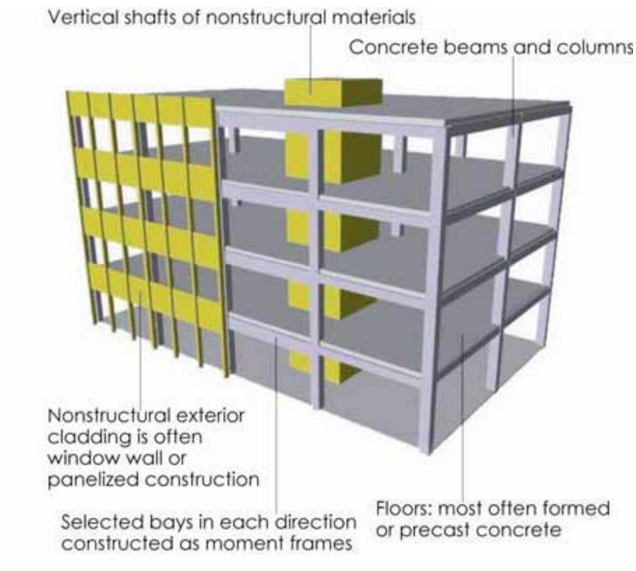


Figure 2.2 Illustration of the C1 building, after FEMA 547 (FEMA 2006).

2.2.2. Reinforced-concrete frame buildings with soft story (C1a)

The C1a buildings are similar to the C1, however, one of the stories has a significantly lower lateral stiffness and strength compared to adjacent floors. This leads to demand concentration in that weak (soft) story, this causing an early collapse, while the remaining stories experience little-to-no deformation and usually remain elastic. Such kind of buildings is very common in Palestine, the infill walls are removed from the first story to be used as a parking space or commercial stores. Figure 2.3 provides a schematic view of the C1a building.

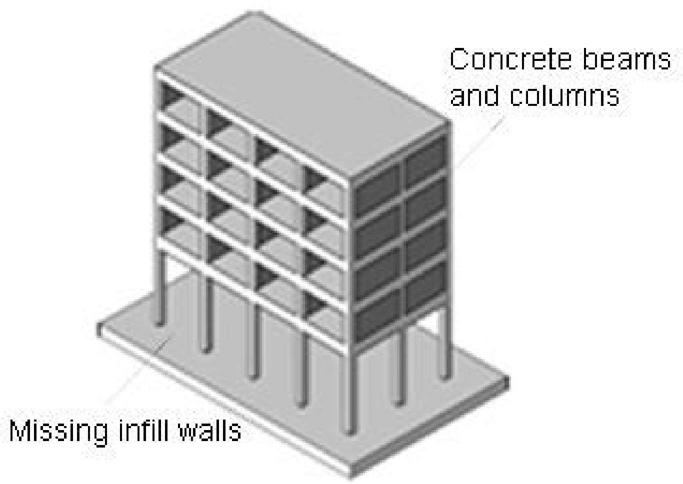


Figure 2.3 Illustration of the C1a building, after FEMA 547 (FEMA 2006).

2.2.3. Shear-wall Building (C2)

C2 buildings are composed of beam-column moment framing systems that primarily support gravity loadings. Those frames are attached to reinforced-concrete shear walls that resist the earthquake-induced loadings and provide the structure with its overall strength and stiffness as indicated by Figure 2.4. The shear walls in C2 buildings take at least 75% of the lateral load.

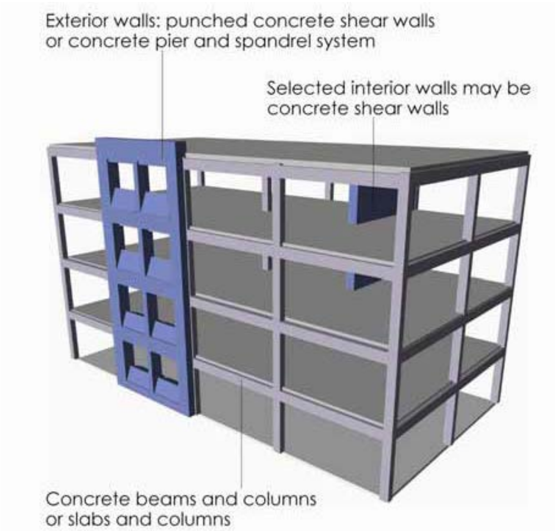


Figure 2.4 Illustration of the C2 building, after FEMA 547 (FEMA 2006).

2.2.4. Unreinforced masonry buildings (URM)

The URM buildings are mainly composed of unreinforced masonry bearing walls, usually located around the perimeter. The bearing walls are often double leaf with plain concrete (or mortar) for filling the cavity in between. Floors are case-in-situ concrete supported by steel joists. The URM building typology is schematically shown in Figure 2.5.

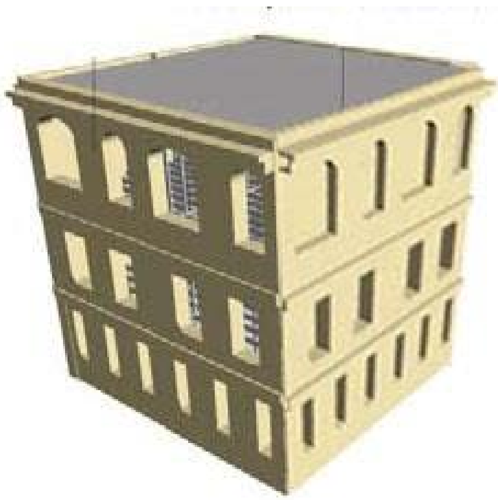


Figure 2.5 Illustration of the URM building, after FEMA 547 (FEMA 2006).

2.3. Applicable Retrofitting Techniques for Common Building Typologies

The current section places emphasis on the applicable retrofitting techniques for the structural deficiency classes identified in section 2.1.1, considering the building typologies highlighted earlier. It is important to understand that not all retrofitting techniques are discussed in detail as many of them are not used in the typical construction practice in Palestine.

2.3.1. Retrofitting strategies of concrete moment frames (C1)

As described earlier, the C1 buildings consist of framing systems that form their stiffness and strength from the rigid connections between beams and columns. C1 buildings in Palestine are generally infilled with masonry walls (with stone cladding at the external walls), which influence the response of the frame. Such walls are disregarded in the structural design process in most of the cases, and they are considered solely as external load rather than a part of the lateral load-resisting system.

Older C1 buildings are often not designed to resist seismic loadings. They possess a very low ductility due to poor confinement, thus leading to inability of sustaining large deformation levels and faster degradation of material strengths. Shear failure is also likely to occur as the non-linear behavior could be controlled by shear yielding (hinging) rather than flexural yielding due to the lack of implementation of the capacity design rule. Moreover, beams are usually designed to be, flexural wise, stronger than columns (i.e., strong-column-weak-beam concept), which allows the formation of plastic hinges in the latter members. This causes a lateral instability and early collapse of the frame as the failure of columns means that the frame will not be capable of withstanding gravity loads.

On the other hand, semi-ductile C1 buildings can be also found, which possess some, but not all, seismic design provisions found in the current codes, standards, and guidelines. Those generally perform better compared to older ones, especially if the structural members (mainly columns) are controlled by flexural behavior. However, such frames might not have sufficient ductility capacity to sustain strong earthquake-induced demands, thus they require retrofitting.

The rehabilitation strategies for the C1 buildings, depending on the deficiency categories and the type of weakness, are summarized in Table 2.1. It can be noticed that many rehabilitation strategies (or techniques) can be adopted to address multiple deficiency categories. Detailed explanations are provided later on some of the rehabilitation techniques that are common in the Palestinian construction industry, such as jacketing, FRP, shear walls, and bracing.

Table 2.1 Retrofitting techniques applicable to the C1 buildings – Reinforced Concrete Frames, after FEMA 547 (FEMA 2006).

Category	Deficiency	Rehabilitation Strategy				
		Add new elements	Enhance existing elements	Improve connections between elements	Reduce demands	Remove selected components
Global strength	Insufficient number of frames or weak frames	Shear walls Steel bracing Concrete frame Steel frame	Increase size of beams and/or columns (concrete and steel jacketing)		Base isolation Damping devices Remove upper stories	
Global stiffness	Insufficient number of frames or frames with low stiffness		Increase size of beams and/or columns (concrete and steel jacketing) FRP wrapping		Damping devices	Remove components creating short column
Configuration	Torsional layout	Shear walls Steel bracing Moment frames		Diaphragm chords		
	Re-entrant corner					
Component (member) detailing	Weak column and strong beams		Concrete jacketing Steel jacketing			
	Inadequate shear strength		Concrete jacketing Steel jacketing FRP wrapping			
	Inadequate confinement or lap splice					
Diaphragms	Inadequate in-plane shear strength	Shear walls Steel bracing Moment frames	Reinforced concrete topping FRP overlay			
	Excessive stresses near openings and/or irregularities	Steel bracing	Reinforced concrete topping FRP overlay			Fill the openings

2.3.1.1. Add new elements: steel bracing

In this rehabilitation technique, steel diagonal braces are added to the existing structures to provide more lateral strength and stiffness. Adding such kind of elements does not usually increase the structural weight significantly. The braces could be in the form of concentric braced frame (CBF), which is the most common type. It is also possible to use eccentric braced frames (EBF), however, they are uncommon due to high costs and difficulty in implementation and detailing. Accordingly, only the CBFs are discussed and explained in this document. Figure 2.6 illustrates the concentric braces, which can be implemented as single diagonal braces, or cross (X) braces (i.e., double braces).

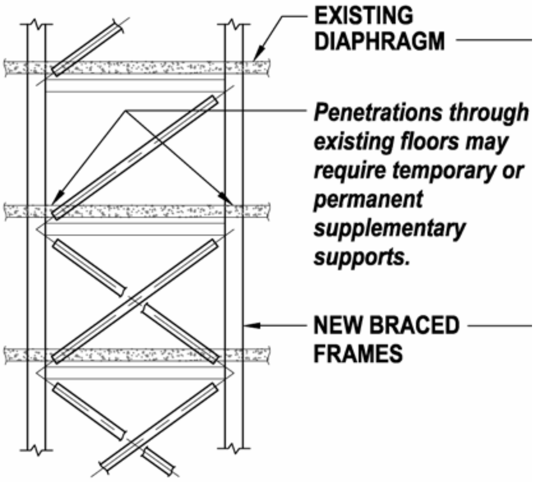


Figure 2.6 Schematic view of the concentric single and double steel braces, after FEMA 547.

Design Considerations:

1. *Modelling assumptions:*

The design and modelling of the lateral force-resisting system, in the case of steel braced frames retrofitting, must account for the strength and stiffness of both the existing reinforced-concrete members and the newly added steel bracing.

2. *Braced frame-concrete interaction:*

The design of steel bracing will be usually governed by maintaining the lateral interstorey drifts within the acceptable range for the existing reinforced-concrete members. This means that the numerical model must account for both the stiffness of the steel braced and concrete frames. However, some engineers might prefer to consider only the steel braced frames as the lateral force-resisting system, determine the drift demands for that system, and then compare with the acceptable limits for the existing concrete frame.

3. Braced frame location:

The new braces are usually located at the exterior of the building as it allows for easier construction and more accessibility, but the new system will be visible and exposed to the environment. Alternatively, the new braces could be installed as an adjacent new construction tied to the existing building. The last option is to install the braces in the internal existing frames, but this would be more difficult to construct and cause more disruption. Typical details of connecting the elements of the steel bracing system to the existing reinforced-concrete members are provided in Figure 2.7, Figure 2.8, and Figure 2.9.

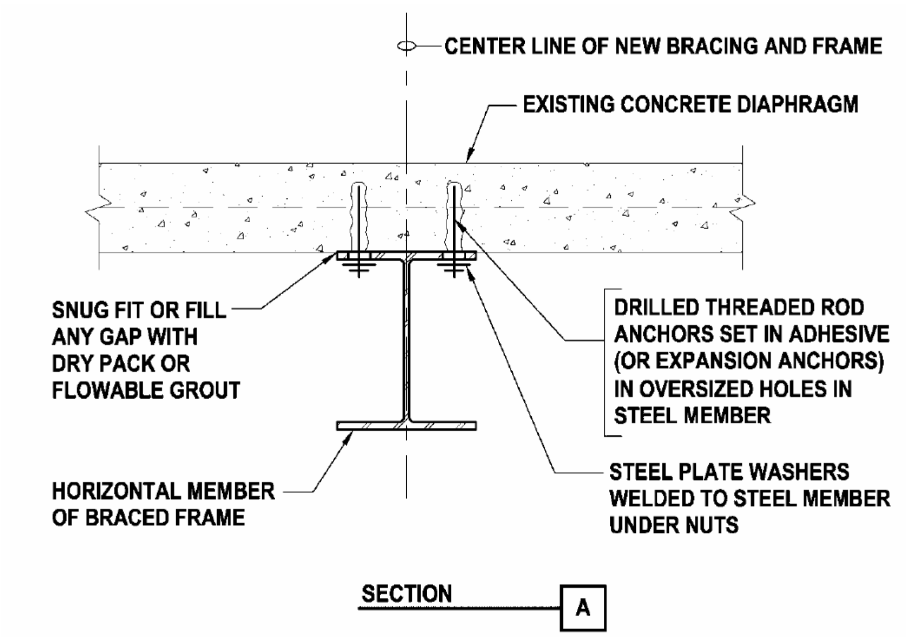


Figure 2.7 Connection between steel horizontal members and concrete diaphragms, after FEMA 547.

4. Braced frame configuration and section type:

In most of the cases where steel braces are needed to increase the lateral strength and stiffness of C1 buildings, a complete system composed of steel horizontal beams and vertical columns, in addition to the bracing elements, is needed. Installing and connecting the steel braces directly to the existing concrete members is not recommended as the transfer of large axial force from those members to the braces via a local connection with limited anchors is not feasible. If the bracing system to be installed includes multiple floors, then the vertical steel columns must be installed continuously across those floors to maintain the integrity of the bracing system. The horizontal steel beams are usually placed below the floor or roof diaphragms (see Figure 2.7), whereas the diagonal braces are placed according to Figure 2.6.

It is recommended to use X-bracing rather than single one, so that even if the compression brace undergoes buckling, the force in the remaining tension brace is transmitted directly to the tension brace on the opposite side of the beam. Alternatively, it is possible to use buckling restrained braces (BRBs), in which braces are surrounded by unbonded concrete to prevent buckling. Those BRBs act the same in both tension and compression, but they are not common in the Palestinian construction industry. The final configuration of the steel bracing system will be selected based on structural issues, architectural challenges, relative strength, and stiffness. Columns and beams are often selected as W-shaped sections (wide-flange shape). Braces on the other hand could be W-sections, or hollow structural sections (HSS) including pipes, tubes, double channels, or angles.

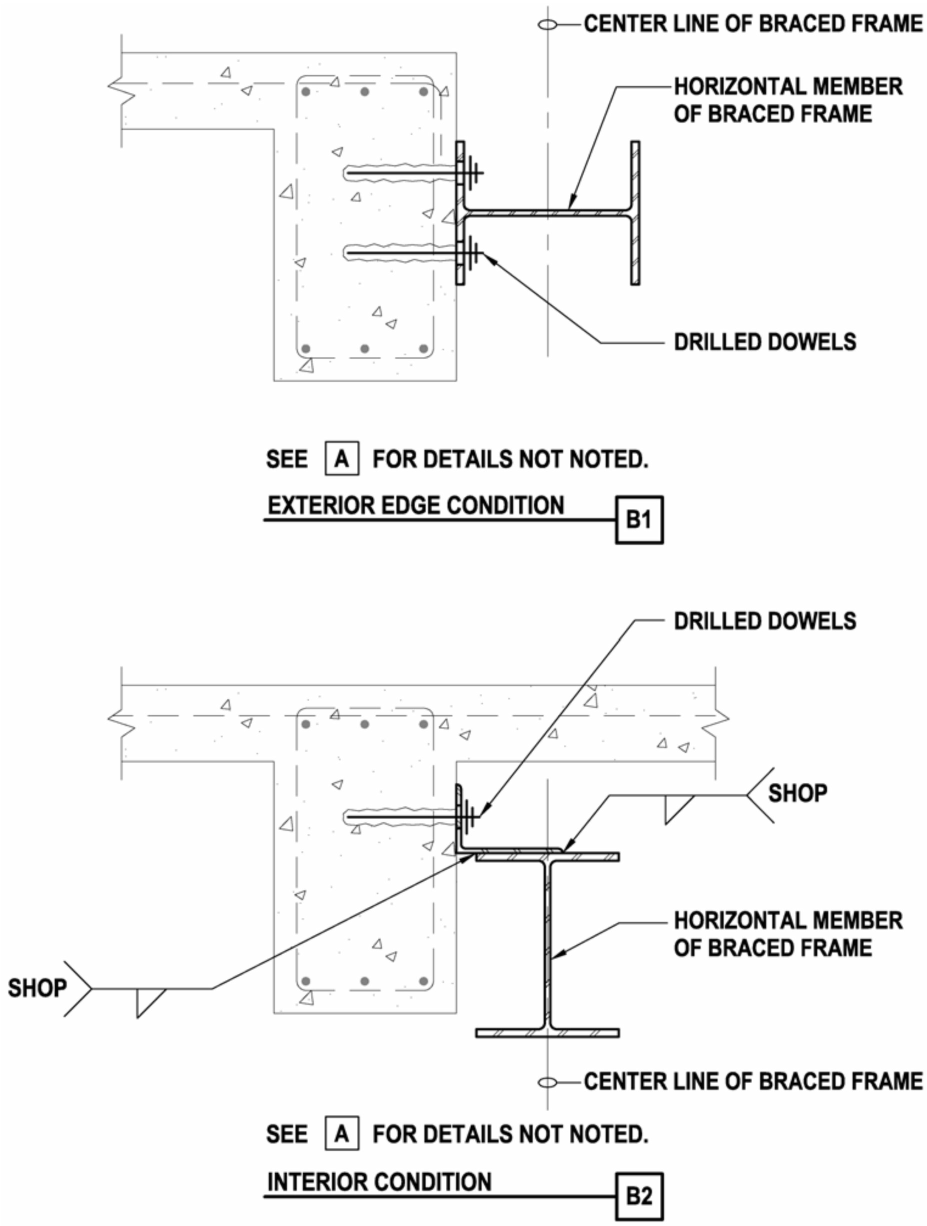


Figure 2.8 Connection between steel members and existing concrete beams, after FEMA 547.

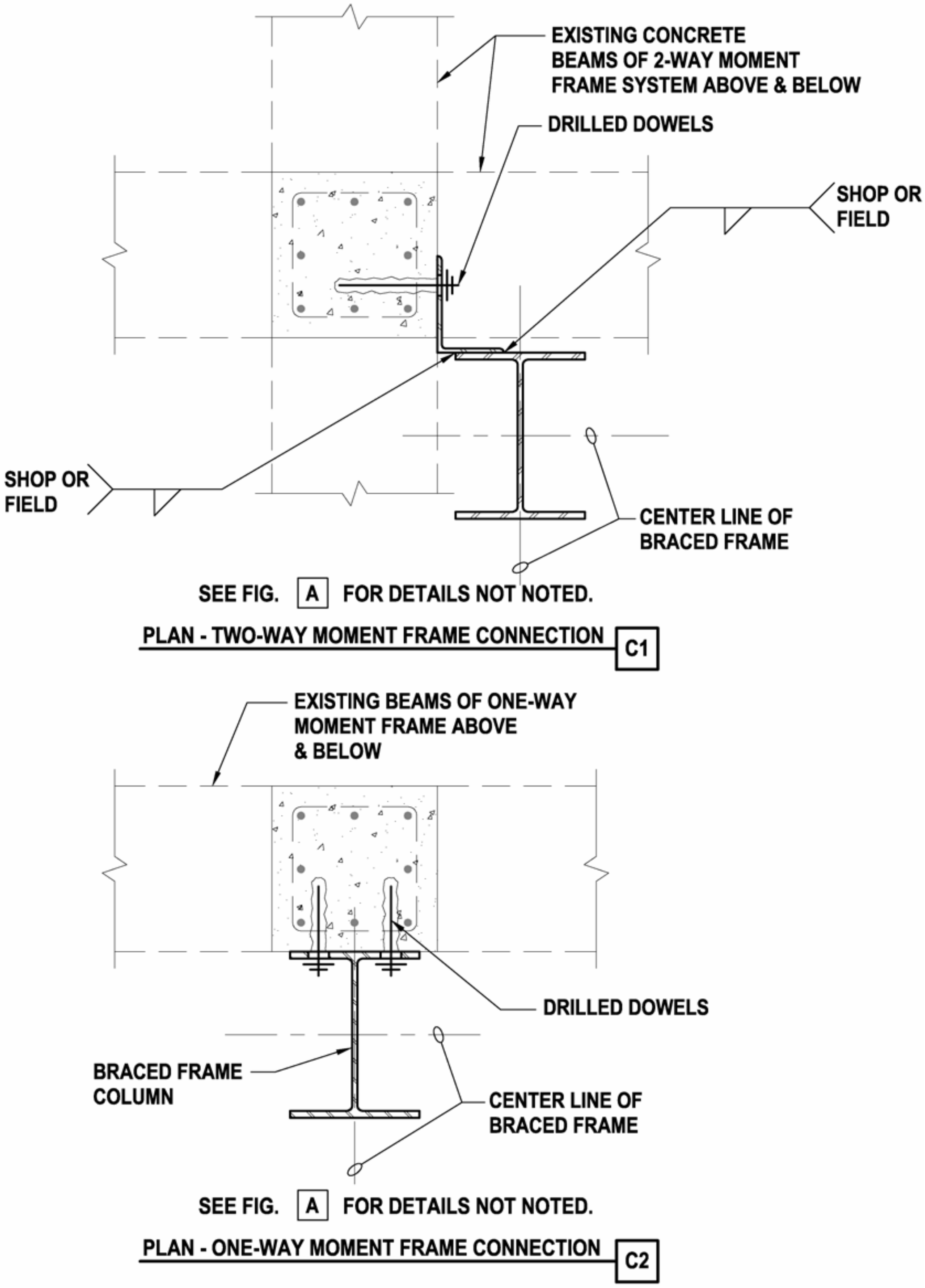


Figure 2.9 Connection between steel members and existing concrete columns, after FEMA 547.

<u>Detailing Considerations</u>:

1. Connection to the existing floors or diaphragms:

The primary concern in connecting the new steel beams to the existing floors or diaphragms is the transfer of large shear force from the diaphragm to the bracing system through relatively localized connections using anchors or bolts. Generally, at least one row of concrete anchors is needed as per Figure 2.7. Classic anchors can be used, which represent threaded rods set in epoxy. It is possible to use drilled expansion anchors if they provide sufficient force transfer.

2. Connection to the existing moment frames:

Steel braced frames are commonly located on or alongside the existing concrete moment frame lines to allow for better use of the existing beams to deliver diaphragm forces to the bracing system. It is generally more desirable to locate the new braces alongside the existing moment frame rather than using them as "infills" within the width of existing concrete beams and columns as the later procedure will make it almost impossible to transfer the large axial loads to the braces via relatively small and localized connections as mentioned previously. If a steel bracing system is installed alongside the existing concrete members, then the anchors can be installed on the side of the adjacent concrete beam as shown in Figure 2.8.

3. Footings:

The addition of a new braced frame to the existing C1 building will require adding a new foundation in most of the cases, or at least augmentation with existing ones. This must be taken into account during the design process of steel bracing.

Cost/Disruption Considerations:

The cost and disruption associated with the installation of a new steel bracing system are generally less than those related to the installation of new shear walls. The bracing could be also easier to construct, and it requires less penetration or drilling compared to concrete shear walls. Moreover, it might not be necessary to prepare the surface of existing concrete members, especially if the bracing system will be installed alongside the structure.

2.3.1.2. Add new elements: concrete shear walls

It is a very common rehabilitation technique that adds significant strength and stiffness to the existing building. The shear walls could be cast-in-place concrete, shotcrete, or fully grouted concrete masonry unit. The former type is the most common one in the Palestinian construction industry, therefore, it is discussed in detail in the current document.

Design Considerations:

1. Frame-wall interaction:

The design of shear walls, similar to the braced frames, will be governed by the controlling the drift demands to be within the acceptable range of limits for the existing concrete members. The structural model in this case must account for the strength and stiffness of both the shear wall and existing concrete frame, and then verify that the drift limits are met with respect to the latter frame. Some engineers might prefer to incorporate the shear walls as lateral force-resisting systems, which is acceptable. In that case, however, the drift demands must be compared with the limits related to the existing concrete frame.

2. Frame-wall configuration:

One of the primary considerations in the design process is to determine whether the existing concrete frame may be used as an effective part of the lateral force-resisting system, or not. This depends on the strength and quality of detailing of the existing concrete members. Such critical considerations must be taken into account to be able to determine the following aspects:

- Shall the shear walls be placed within the plane of the existing frames?
- Shall the shear walls be built as vertical continuous elements then joined to the main existing concrete frame?
- Shall the shear walls be constructed as separate vertical elements, i.e., independent of the existing concrete frames?

More discussion is provided later about the choice of designing and constructing shear walls based on the condition of the existing concrete frame members.

3. Wall location:

New shear walls might be placed on the exterior or interior of the existing structure. In the former case, the walls are easier to construct, and they have more accessibility and less cost. But the visibility of the wall might impact the aesthetics of the existing structure. Interior walls on the other hand are usually located along the frame lines, specifically at the moment frame bays. Beams that frame directly into the ends of the new walls might act like coupling beams.

<u>Detailing Considerations</u>:

1. Connection to existing concrete floors and roof diaphragms:

Care must be exercised when it comes to the connection between the top of the new shear wall and the bottom of the existing frame member (i.e., diaphragm or floor). The construction joint here must be tight, without any gaps, to ensure a full transfer of shear forces without slippage. Typical details of the connection between cast-in-place shear walls and concrete solid slabs are provided in Figure 2.10 and Figure 2.11. If the diaphragm consists of ribbed/waffle/joist slabs, then the detailing of the connection must be in accordance with Figure 2.12.

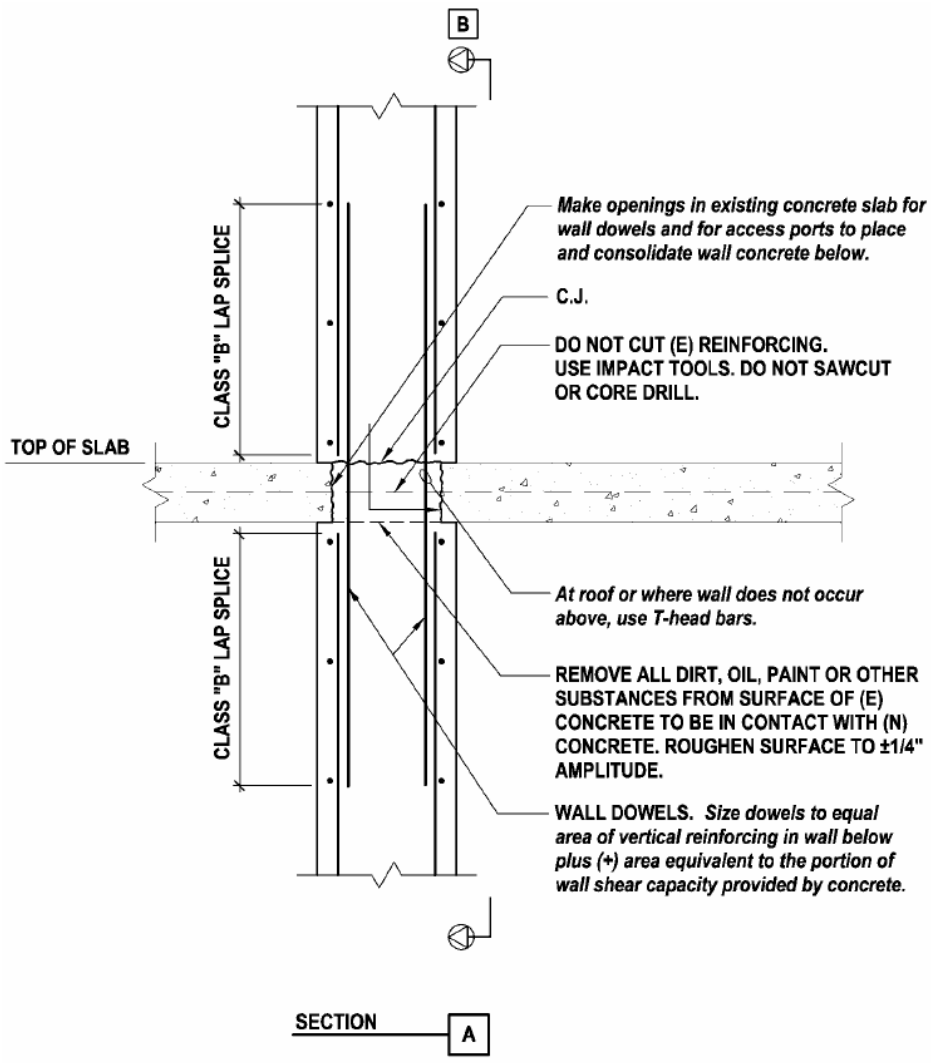


Figure 2.10 Connection between a concrete wall and a solid slab, after FEMA 547.

If the new shear wall is perpendicular to the direction of the joists or ribs, then the slab can be removed between the ribs/joists as illustrated in Figure 2.13. As the installation of continuous horizontal steel bars is not possible through the perpendicular ribs, installation of horizontal loop ties may be necessary at the upper portion of the shear wall between the ribs.

2. Connection to existing frames:

New shear walls are usually placed on or alongside existing frame lines to allow for better use of the existing beams (as diaphragm collectors) and existing columns (as chords or boundary elements). It is always desirable to locate the walls alongside the existing frames rather than using them as "infills" within the existing frame panels. In the former case, the wall diaphragm connections can be designed as explained previously in Figure 2.10 to Figure 2.13. However, in the "infill" configuration, vertical wall dowels must be threaded through densely reinforced beams, and concrete placement becomes much more difficult.

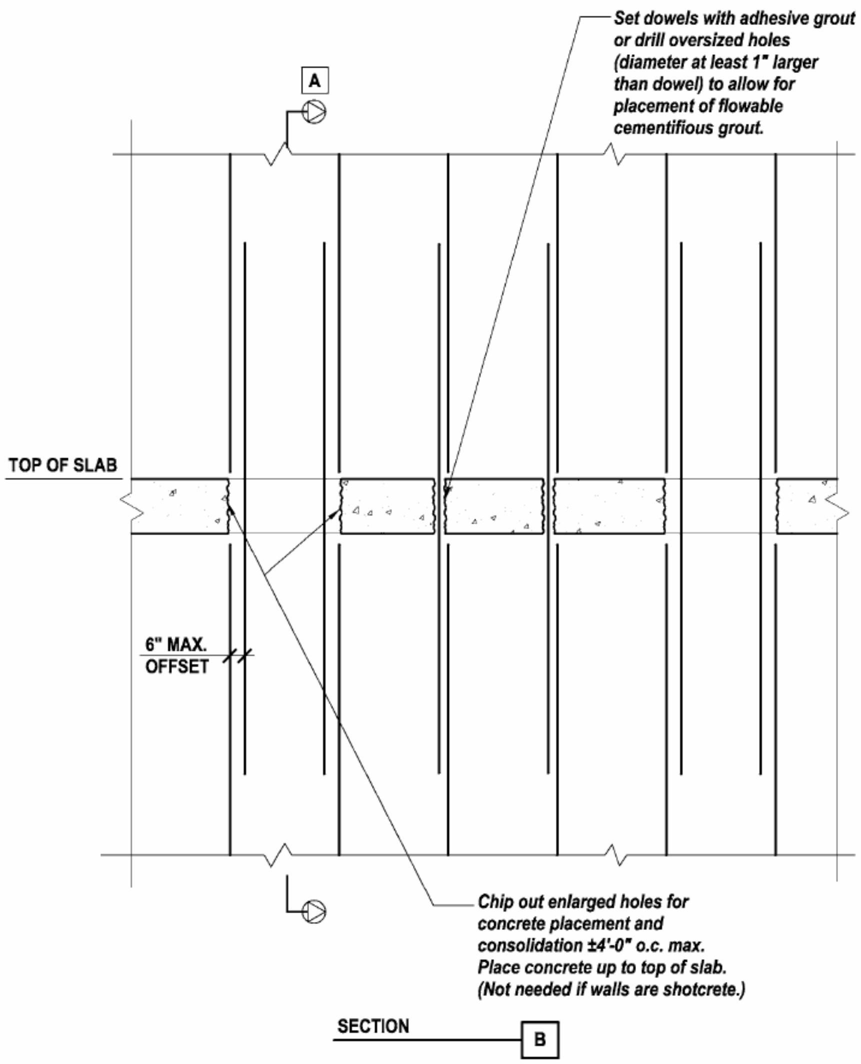


Figure 2.11 Connection between a concrete wall and a solid slab – partial elevation, after FEMA 547.

If the existing columns have sufficient strength and appropriate structural detailing, they might be considered as wall chords (or boundary elements) by simply dwelling them into the new shear walls. However, existing columns usually require strengthening, otherwise, new wall chords (or boundary elements) might be needed.

3. Installation of additional collectors:

Installation of shear walls to an existing frame usually results in an increased diaphragm demands at the individual walls. One of the advantages of placing the new walls at an existing frame line is that the existing beams can be used as collectors. However, such collectors might require strengthening due to insufficient reinforcement and concentrated stresses.

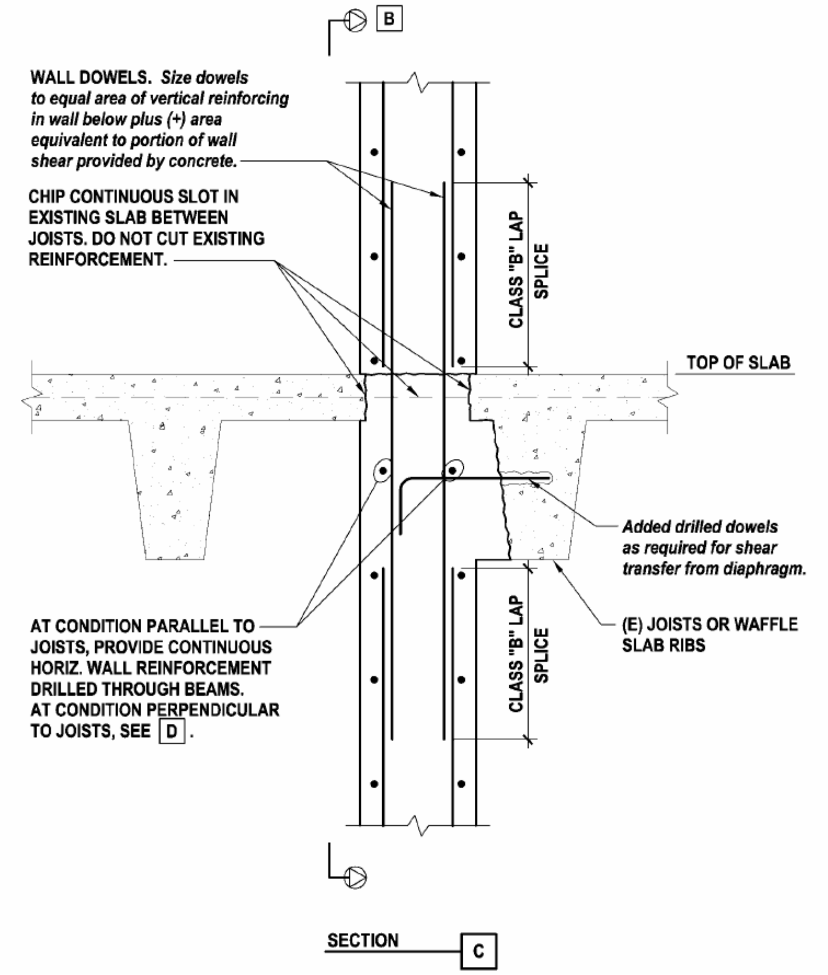


Figure 2.12 Connection between a concrete wall and a ribbed/joist/waffle slab, after FEMA 547.

4. Footings:

New shear walls require the installation of new footings on almost all the cases, or at least augmentation of existing ones, in order to support the additional weight of the walls and prevent any overturning due to lateral loads. If the new walls are located in between frame lines, new foundations might be used to engage more than one column.

Cost/Disruption Considerations:

The construction of new shear walls is generally very disruptive to building occupants, especially with the excessive vibration, noise, and dust. Cost wise, it is considered more expensive when compared to installing a steel braced frame.

Construction Considerations:

The existing concrete surfaces that are in contact with the new walls must be first cleaned from all finishings and then roughened in order to provide an appropriate interlock between the new and existing concrete surfaces.

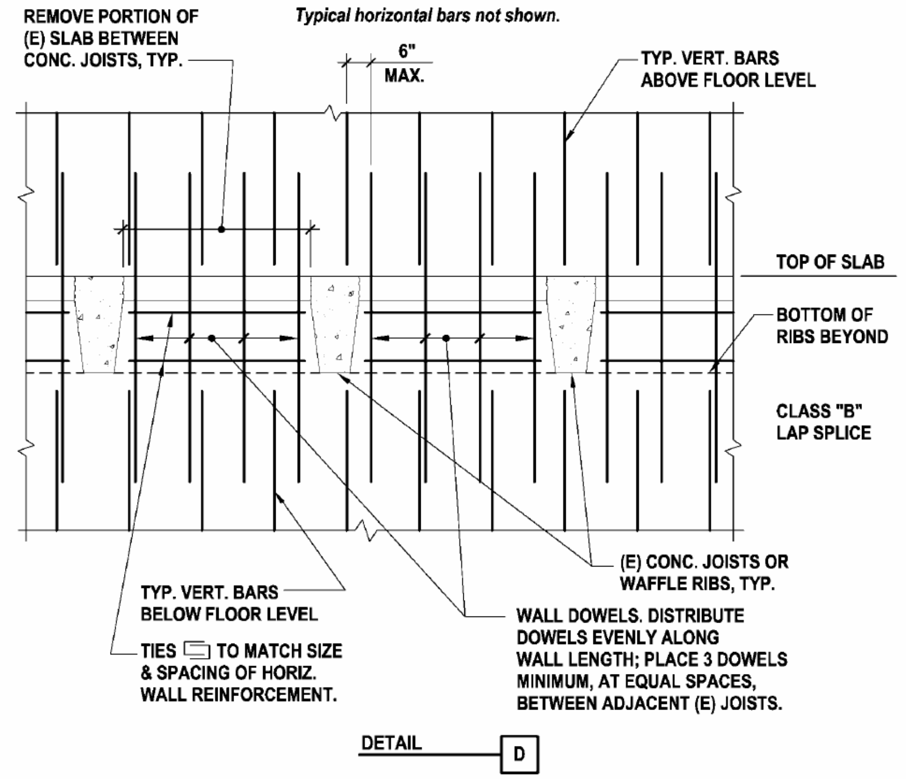


Figure 2.13 Connection between a concrete wall and a perpendicular rib/joist, after FEMA 547.

2.3.1.3. Enhance existing columns: fiber-reinforced polymers (FRP)

The fiber-reinforced polymer (FRP) overlay is an effective method in both building and bridge construction. Columns are wrapped with unidirectional fibers in the horizontal direction, thus providing additional shear strength and confinement, similar to that provided by hoops and stirrups. The confinement in turn offers clamping action that improves the lap splicing areas, enhances the compression characteristics of concrete, and increases ductility capacity. It should be noted that FRP wrapping does not improve the flexural strength of columns, therefore, they should not be used for such a reason.

Design and Detailing Considerations:

The primary deficiencies in a column are usually as follows: 1) lack of shear strength; 2) insufficient ductility capacity; 3) inappropriate lap splice. The FRP overlays in those cases will enhance the stress-strain behavior of concrete through additional confinement. It should be noted that the confinement applied by FRP wrapping on circular sections is much more efficient than that applied on rectangular sections. This is because the radial passive pressure due to confinement acts over the full section perimeter in the former case, whereas it acts on the corners only in the latter case. Therefore, it is not recommended to apply FRP wrapping for columns with more than 1.5 of depth-to-width ratio.

The increase in stiffness and flexural strength of the column due to FRP wrapping is marginal and can be generally disregarded. It basically stems from the increased concrete compressive strength for both the cover and core. This causes the bending neutral axis depth to reduce, thus allowing a bigger lever arm and larger flexural strength accordingly. The final design thickness of FRP wraps can be determined depending on the type of deficiency. In other words, it is possible to calculate a thickness to account for additional shear strength, or another thickness to address the lap splice issues, or a thickness to account for ductility capacity and enhance the compression strength. The maximum of any of those thickness values must be used as the final thickness value of the FRP wrapping. Design equations can be found in detail through the ACI 440 (ACI 440.2R 2008).

Figure 2.14 illustrates the typical layout of FRP wrapping around the column based on the type of the deficiency (i.e., compression, lap splice, shear strength). Figure 2.15 on the other hand shows typical cross sections of circular and rectangular concrete columns wrapped with FRP layers. Some typical details are also shown. It should be noted that a gap of at least 10 mm must be left between the FRP wrapping and the edge of the concrete column (i.e., slabs, beams, footings) to prevent the bearing action that can lead to unnecessary increase in the flexural strength and stiffness of the column.

Cost/Disruption Considerations:

The overall costs of retrofitting using FRP wrapping in comparison with more traditional techniques depend on several factors such as the cost of raw materials and specialized labor. The cost of quality control and equipment also has an important contribution. Therefore, cost-benefit analysis must be performed by the engineer to assess which retrofit technique is the most economically feasible. Generally, the FRP wrapping costs more than traditional steel or concrete jacketing, but it has the advantage of being the least disruptive technique amongst all of them. Practically, it has no effect on the aesthetics or the architectural configuration of the existing reinforced concrete columns.

Construction Considerations:

It is understood that the access around the columns is usually prevented by the presence of infill walls and partitions, in addition to ceilings. This requires making a localized gap to allow full access around the entire perimeter of the column.

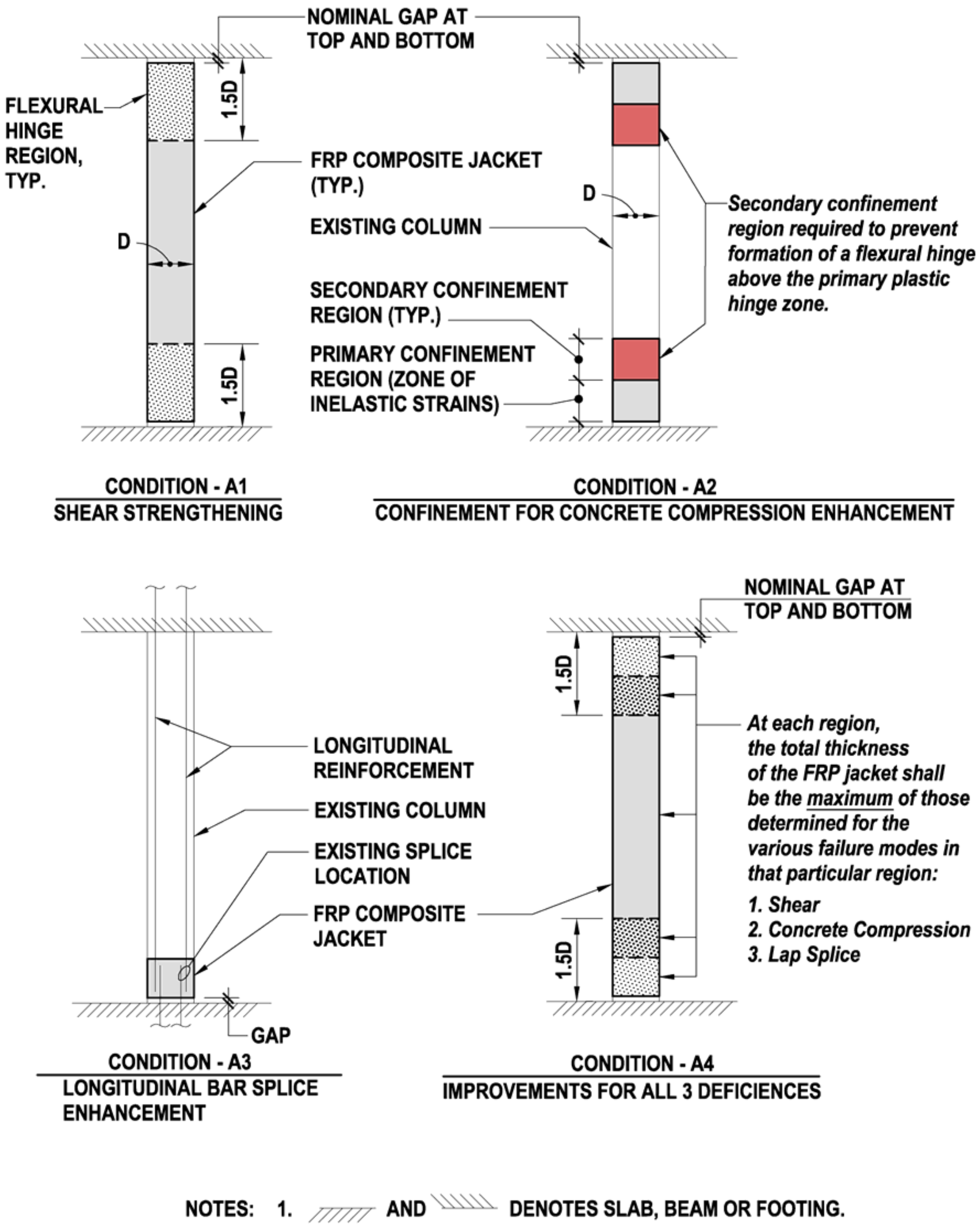


Figure 2.14 Layout of FRP wrapping depending on the deficiency, after FEMA 547.

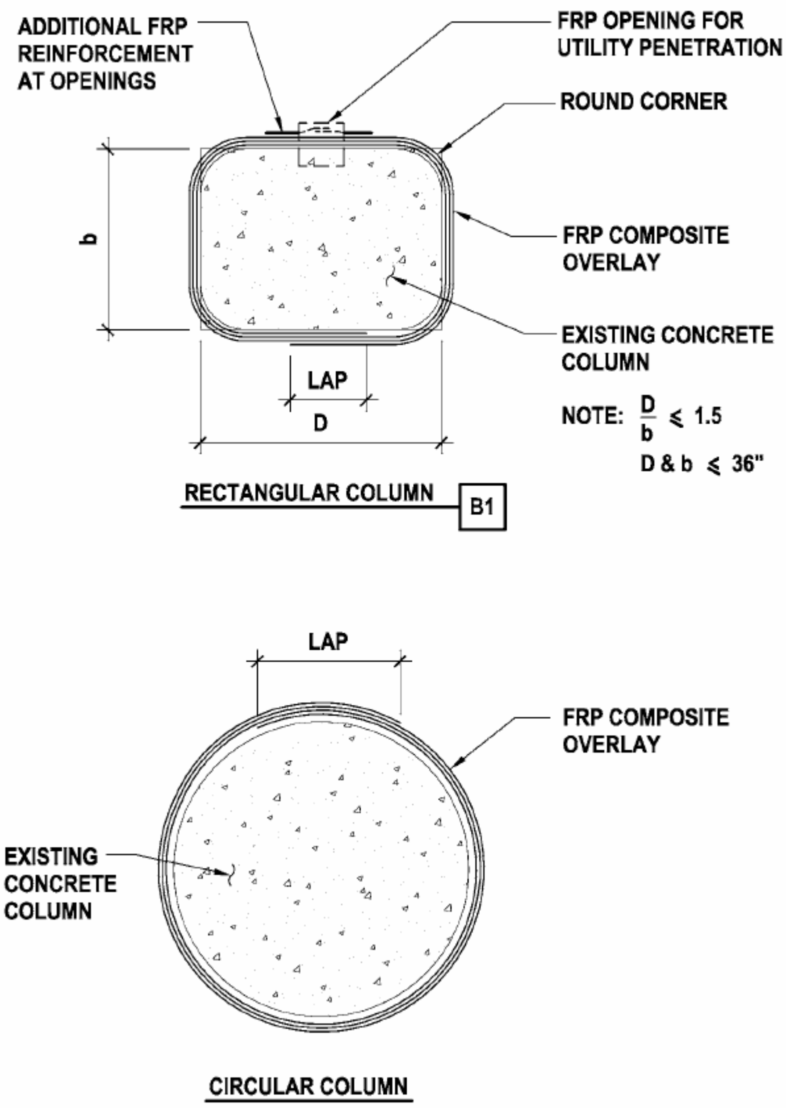


Figure 2.15 FRP wrapping around circular and rectangular column sections, after FEMA 547.

2.3.1.4. Enhance existing columns: steel jacketing and concrete jacketing

Installing concrete or steel jacketing is a more traditional retrofitting technique for enhancing the performance of existing concrete columns, compared to FRP wrapping. Concrete jacketing improves both flexural and shear strength, in addition to enhancing ductility capacity, stiffness, and confinement. Steel jacketing on the other hand has the same effects as concrete jacketing, but it does not improve flexural strength because a gap is usually left between the edge of the jacket and the boundaries of the column to prevent bearing action. Figure 2.16 provides typical cross-section details for existing columns retrofitted with concrete jackets, rectangular steel jackets, and elliptical steel jackets.

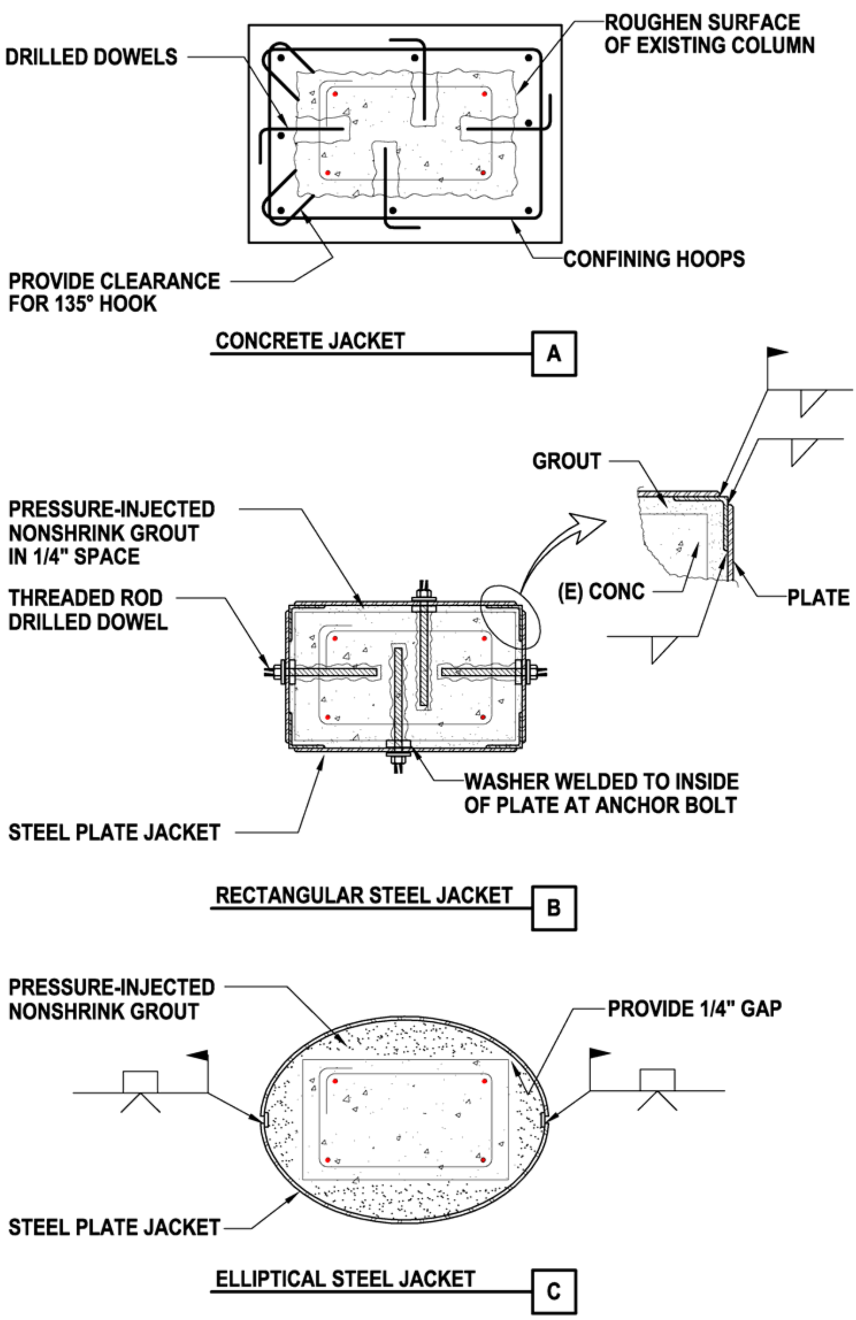


Figure 2.16 Concrete jacketing and steel jacketing (rectangular and elliptical), after FEMA 547.

<u>Design and Detailing Considerations</u>:

For concrete jackets, the design can be performed using the traditional procedures provided by the ACI 318 (ACI Committee 318 2014) guidelines. However, several critical points must be taken into account:

- It is possible to assume that the column with concrete jacket acts like a monolithically casted column. However, quality control must be assured on site to allow appropriate bonding between the existing column and new jacket. This is done by roughening the surface of the existing column and using sufficient number of drilled dowels to achieve a fully composite action between the jacket and the existing column.

- Upon installing the concrete jacket, it should be assumed that the confinement and shear strength are provided by the hoops of the jacket. In other words, the stirrups of the existing column may be neglected.
- To account for any inappropriate bonding between the jacket and existing column, it is possible to assume the compressive strength of the existing column for the entire section including the new jacket, as done in Aljawhari et al. (2022).
- The concrete jacketing improves flexural strength only if the longitudinal steel bars are extended through different floors and foundations.

In the new concrete jackets, 4 longitudinal bars must be at least provided (at each corner). 135-degree hooks must be also installed, and this might govern the thickness of the jacket.

For steel jackets, elliptical ones are much more efficient with respect to confinement compared to rectangular jackets as the radial passive pressure acts on the corners only in the latter case (similarly to the FRP wrapping). Therefore, rectangular steel jacketing is not recommended if the aspect ratio of the cross section is high. If elliptical steel jackets are used, the corners of the concrete column must be trimmed so that steel can pass by. There must be a gap between the steel jacket and existing column of at least 6 mm. Such a gap must be filled with grout as clarified in Figure 2.16. A gap must be also provided between the ends of steel jackets and column boundaries to permit rotation without any additional bearing as mentioned earlier.

It should be noted that the additional shear strength provided by elliptical jackets can be evaluated using the model proposed by Priestley et al. (1994), as per Eq. (2.1) for the strong direction and Eq. (2.2) for the weak direction of the jacket.

$$V_{j,strong} = 3.46 f_{yj} t_j \left(D_j - t_j\right)[1 - (1 - 0.25\pi)\, B_j/D_j] \tag{2.1}$$

$$V_{j,weak} = 3.46 f_{yj} t_j \left(B_j - t_j\right)[1 - (1 - 0.25\pi)\, D_j/B_j] \tag{2.2}$$

where $V_{j,strong}$ and $V_{j,weak}$ are shear strength provided by the elliptical steel jackets in strong and weak directions, respectively. f_{yj} is the yield strength of steel jacket, t_j is the thickness of jacket, D_j is the long principal diameter of the steel jacket, and B_j is the short diameter of the short principal diameter of the steel jacket, as schematically illustrated in Figure 2.17.

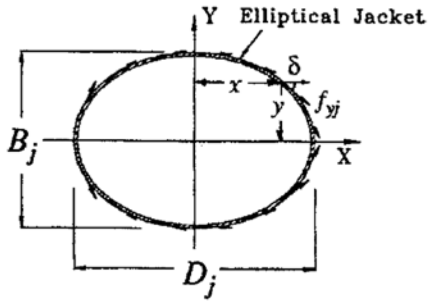


Figure 2.17 Geometric features of steel elliptical jacket, after Priestley et al. (1994).

<u>Cost/Disruption Considerations</u>:

Generally, traditional retrofit techniques like concrete and steel jackets are less with respect to cost when compared to FRP wrapping. However, engineers must take into account the cost of raw materials, specialized labor, equipment, and quality control. Despite the lower cost of steel and concrete jackets, they cause more significant disruption to the architectural configuration as well as the aesthetics of the building compared to the FRP wrapping.

<u>Construction Considerations</u>:

Concrete jackets are typically performed using cast-in-place concrete, rather than shotcrete. The need of building formwork is one of the major disadvantages of such a retrofit technique, when compared to steel jackets or even FRP wrapping. Placing the concrete and vibrating it is also very challenging. Steel jackets on the other side are easier to construct, but they might be quite heavy, and this makes the lifting and accessibility very difficult. Sometimes, it might be necessary to break down the steel jackets into several pieces so that they are assembled together on site. This requires additional costs and efforts as a field welding crew becomes a must.

2.3.2. Retrofitting strategies of concrete moment frames with soft story (C1a)

The C1a buildings are similar to typical C1 buildings. The main difference is the presence of a soft story in the C1a buildings, which has significantly lower strength and stiffness compared to adjacent stories, making it susceptible to early failure to the concentration of deformation and stress. The retrofitting techniques for the C1a buildings are almost similar to the case of C1 buildings. The difference is that the main target of retrofitting is fixing the soft story mechanism through mainly increasing the lateral strength and stiffness of that story.

The most effective retrofitting strategy to fix the soft story mechanisms is through adding new elements, such as concrete shear walls and steel bracing. Those increase the lateral strength and stiffness significantly, and they can easily shift the soft story mechanism. However, such techniques are quite costly and might cause major disruption to occupants and aesthetics of the building. Therefore, it is possible to enhance the performance of the existing columns in the soft story by using concrete jacketing, which improves stiffness, flexural strength, and shear capacity. It is also less disruptive and more economically feasible.

It is quite important to understand that using steel jacketing or FRP wrapping to improve the soft story mechanism is not desirable. This is because such techniques are only effective in improving shear strength, confinement and ductility capacity. Conversely, fixing the soft story mechanism might require a major improvement in the flexural strength, which cannot be achieved through the FRP wrapping and steel jacketing (recall that a gap must be left between the end of FRP wrapping or steel jackets and the column boundary to prevent any bearing that could lead to increasing flexural strength).

2.3.3. Retrofitting strategies of shear-wall buildings (C2)

The lateral load in the C2 buildings is resisted by the dual action of shear walls and traditional moment frames (beams and columns). In those buildings, shear walls take at least 75% of the lateral load due to their significantly larger stiffness and strength. Some buildings might contain incidental shear walls with very small cross sections and low stiffness. Such buildings must be classified as C1 rather than C2 during the retrofit design process.

C2 buildings are generally quite stiff due to the presence of shear walls. Therefore, elastic and early post-yielding response should incur minimal drifts, indicating that the response is most likely satisfactory, even if the building is old. However, the post-yielding response is highly dependent on the detailing, distribution, and characteristics of the shear walls used in the building, and to a lower extent the characteristics of the moment frame.

When the C2 buildings are subjected to increasing lateral load, shear walls will force the yielding first in the spandrels (or coupling beams) that restrict their deformation. The shear walls then will either exercise a rocking mechanism on their foundations, suffer shear yielding and cracking, or alternatively form a flexural hinge at their base. The spandrels (or coupling beams) on the other hand might experience either flexural or shear yielding. In the former case, the lateral strength of the system is maintained. In the latter case, however, a fast degradation might occur if the spandrel is not well detailed, and then the shear walls will start acting like cantilevers from their base. It is desirable to have shear walls with flexural hinging as the strength will degrade only at excessive drift levels, conversely, the shear yielding response leads to a very fast degradation of strength even at low drifts.

Regarding the moment frame in C2 buildings, their contribution to the lateral load-resisting system depends on their detailing and characteristics. It is common for engineers to disregard their effect in the lateral response analysis and assume that they only resist gravity loads.

However, it is always recommended to consider their interaction with the shear walls when building a numerical model for analysis. The rehabilitation strategies for the C2 buildings, depending on the deficiency categories and the type of weakness, are summarized in Table 2.2. It can be noticed that many rehabilitation strategies (or techniques) can be adopted to address multiple deficiency categories. Detailed explanations are provided later on some of the rehabilitation techniques that are common in the Palestinian construction industry.

It can be noticed in Table 2.2 that the majority of retrofit techniques used for the C2 buildings are similar to those used to retrofit the C1 buildings, such as concrete jacketing, steel jacketing, adding steel bracing system, and adding new shear walls. Such techniques are explained earlier in section 2.3.1. Therefore, only the additional retrofitting techniques that differ from those used for C1 buildings and are applicable in the Palestinian construction industry at the same time, are explained in the below sections.

2.3.3.1. Enhance existing walls: FRP wrapping

FRP overlay can be used to enhance the shear strength of a concrete shear wall. The FRP overlay can be applied to one or both sides of the existing wall. Whenever possible, the FRP overlay must be wrapped around the entire wall perimeter to aid in anchoring the overlay itself. The FRP overlay is placed in a way that the unidirectional fibers are pointing towards the horizontal direction. This significantly enhances shear strength and develops a flexural post-yielding behavior for the retrofitted shear wall by changing the response from being governed by shear yielding to flexural hinging. At the coupling beams (spandrels), it is common instead to use vertically oriented fibers. A generic layout for the FRP overlays used to strengthen the shear walls and coupling beams is illustrated in Figure 2.18.

Design and Detailing Considerations:

The contribution to shear resistance by the FRP overlay can be easily evaluated in a similar manner to that used for wall reinforcement. In other words, both the horizontal bars and FRP overlays will resist the shear demands.

Table 2.2 Retrofitting techniques applicable to the C2 buildings – Reinforced Concrete Shear Walls, after FEMA 547 (FEMA 2006).

Category	Deficiency	Rehabilitation Strategy				
		Add new elements	Enhance existing elements	Improve connections between elements	Reduce demands	Remove selected components
Global strength	Insufficient in-plane wall shear strength	Shear walls Steel bracing Concrete frame Steel frame	Concrete/steel jacketing FRP wrapping		Base isolation	
	Insufficient flexural strength		Enhance chords			
	Inadequate capacity for coupling beams		Strengthen beams			Remove beams
Global stiffness	Excessive drifts	Shear walls Steel bracing	Concrete/steel jacketing FRP wrapping		Damping devices	
Configuration	Discontinuous walls	Shear walls Steel bracing	Concrete/steel jacketing for existing columns	Improve connection to diaphragm		Remove wall
	Torsional layout	Add balancing shear walls or steel bracing				
	Re-entrant corner	Add floor area		Provide chords in the diaphragms		
Component detailing	Inadequate wall for out-of-plane bending		Concrete jacketing			
	Inadequate shear strength for the wall		Concrete/steel jacketing FRP wrapping			
Diaphragms	Inadequate in-plane shear strength		Reinforced concrete topping FRP overlay			
	Excessive stresses near openings and irregularities	Add chords				Fill the openings

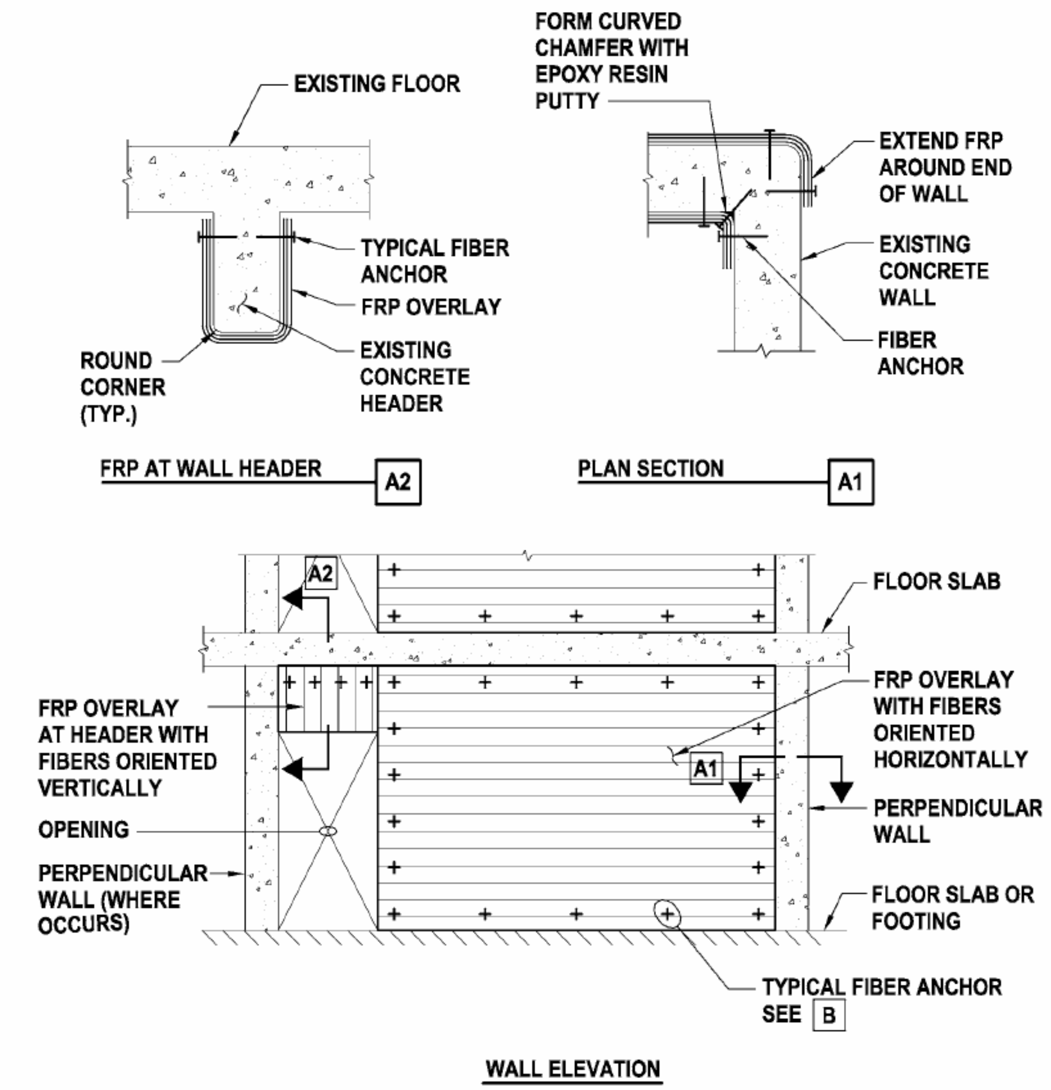


Figure 2.18 Layout of FRP overlays for shear walls and coupling beams, after FEMA 547.

It should be noted that the effective area of FRP per unit width and its contribution to the overall shear strength is limited by the bond and anchorage strength. Practically, engineers use either a single or a double layer of FRP overlays at each side of the wall. Whenever possible, it is always better to wrap the overlays around the entire wall's body to ensure a more efficient bonding and confinement. Typical details on the anchorage of the FRP overlays over the shear walls are provided in Figure 2.19.

The contribution of the FRP overlays to the ductility of the walls depends primarily on the governing sway mechanism of the wall. For shear-dominated walls, where the post-yielding behavior requires slippage at crack locations, the ductility capacity will not be significant. In contrast, a wall dominated by flexural yielding can perform better with respect to sustaining larger deformation levels. Accordingly, a typical goal of FRP overlays is to make the shear wall flexurally-critical.

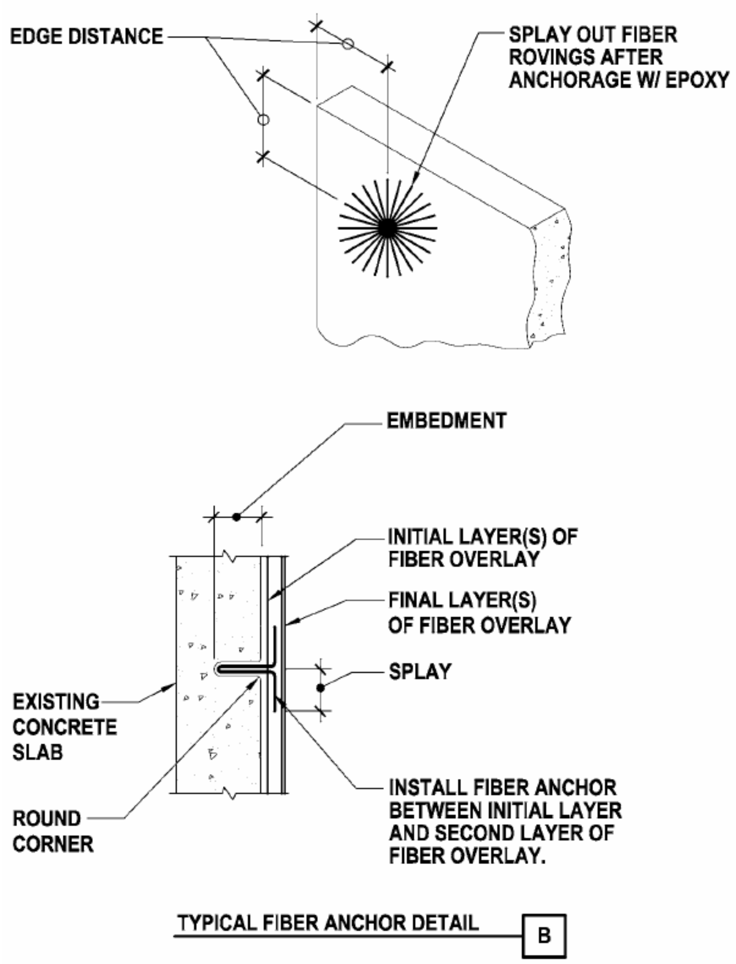


Figure 2.19 Details of anchors used to fix FRP overlays on shear walls, after FEMA 547.

2.3.4. Retrofitting strategies of unreinforced masonry buildings (URM)

The URM buildings are often composed of masonry stone or brick bearing walls located at the building perimeter. Floors are typically made of reinforced-concrete slabs that are either two-way slabs or one-way ones supported by steel joists. Wooden slabs/joists are not common in the Palestinian construction industry, instead, they are widely common in the United States and Europe. Such buildings are expected to perform poorly during earthquakes, with the most common failure mode being the outward collapse of external walls caused by the separation of those walls from the floors and roof diaphragms.

Natural stone is the most common material used as a masonry unit for URM Palestinian structures. The masonry wall in this case is a double-leaf with a cavity filled with mortar or unreinforced concrete. The presence of such cavities could reduce out-of-plane strength of the masonry wall. The in-plane lateral strength, on the other hand, depends significantly on the relative strength of masonry (i.e., stone units) 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. Mortar materials incorporating cement, unlike those containing lime and sand, are usually strong, thus offering the advantage of enhancing the overall lateral strength of the wall. However, this might lead to brittle cracking in masonry units, resulting in lower ductility capacity.

The main deficiencies in URM buildings stem from the unbraced parapets, which represent a falling hazard due to their poor connection between the walls and diaphragms. Poorly connected walls are also another significant issue that can lead to wall failure and loss of gravity-load supporting system, in addition to inadequate out-of-plane strength. Table 2.3 provides the proposed retrofitting strategies with examples for different categories of seismic deficiencies. It is important to note that many of those techniques are previously discussed in detail (see Section 2.3.1 and Section 2.3.3) such as concrete and FRP overlays. The additional retrofitting techniques, which are considered applicable to the Palestinian construction reality, are thoroughly explained in the following sections.

2.3.4.1. Component detailing: parapet bracing or removal

Unbraced parapets are usually the first elements to fall during ground shaking due to inadequate bending strength and ductility. Those parapets can be either removed or braced. Bracing is usually composed of steel angles. The brace is anchored near to the top of the parapet and to the roof. An example of such bracing is provided in Figure 2.20.

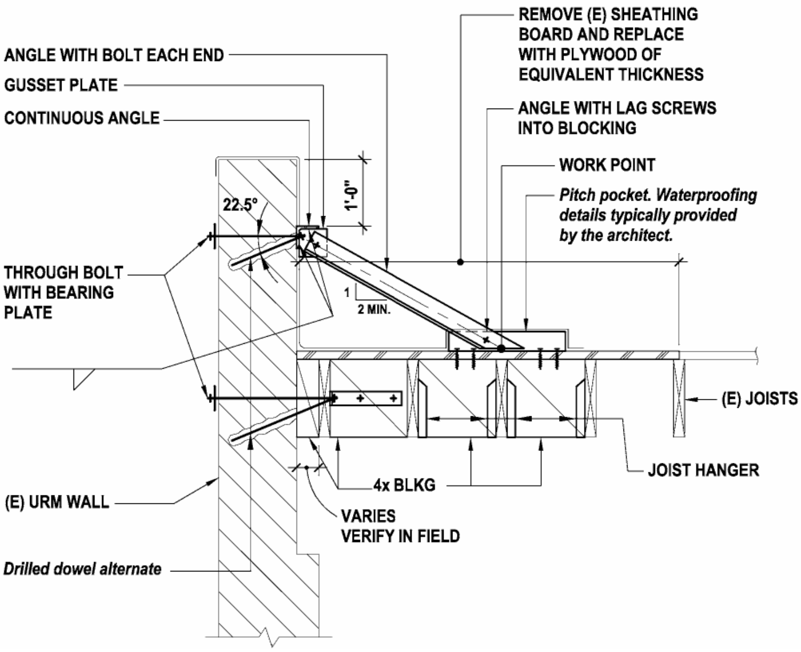


Figure 2.20 Details of parapet bracing in URM buildings, after FEMA 547.

Table 2.3 Retrofitting techniques applicable to the URM buildings – Unreinforced Masonry, after FEMA 547 (FEMA 2006).

Category	Deficiency	Rehabilitation Strategy				
		Add new elements	Enhance existing elements	Improve connections between elements	Reduce demands	Remove selected components
Global strength	Insufficient in-plane wall strength	Shear walls Steel bracing Concrete frame Steel frame	Concrete wall jacketing FRP wall wrapping		Base isolation	
Configuration	Soft story, torsional layout, weak story	Shear walls Steel bracing Concrete frame Steel frame				
Component detailing	Inadequate wall for out-of-plane bending		Reinforced cores FRP wall overlay Concrete wall jacketing			
	Unbraced parapet		Parapet bracing			Remove parapet
	Poorly-anchored veneer or appendages		Add ties			Remove veneers or appendages
Diaphragms	Inadequate in-plane strength and/or stiffness	Shear walls Steel bracing Concrete frame Steel frame	Reinforced concrete topping FRP overlay			
	Excessive stresses near openings and irregularities	Steel braces or steel strap reinforcement				

Another option is to remove the parapet entirely. However, this option will lead to reduction of vertical stress on wall-to-roof anchors. Therefore, removing the parapet is usually combined with installing a concrete cap (or bond beam) as part of the wall-roof anchorage. Details for such actions are illustrated in Figure 2.21.

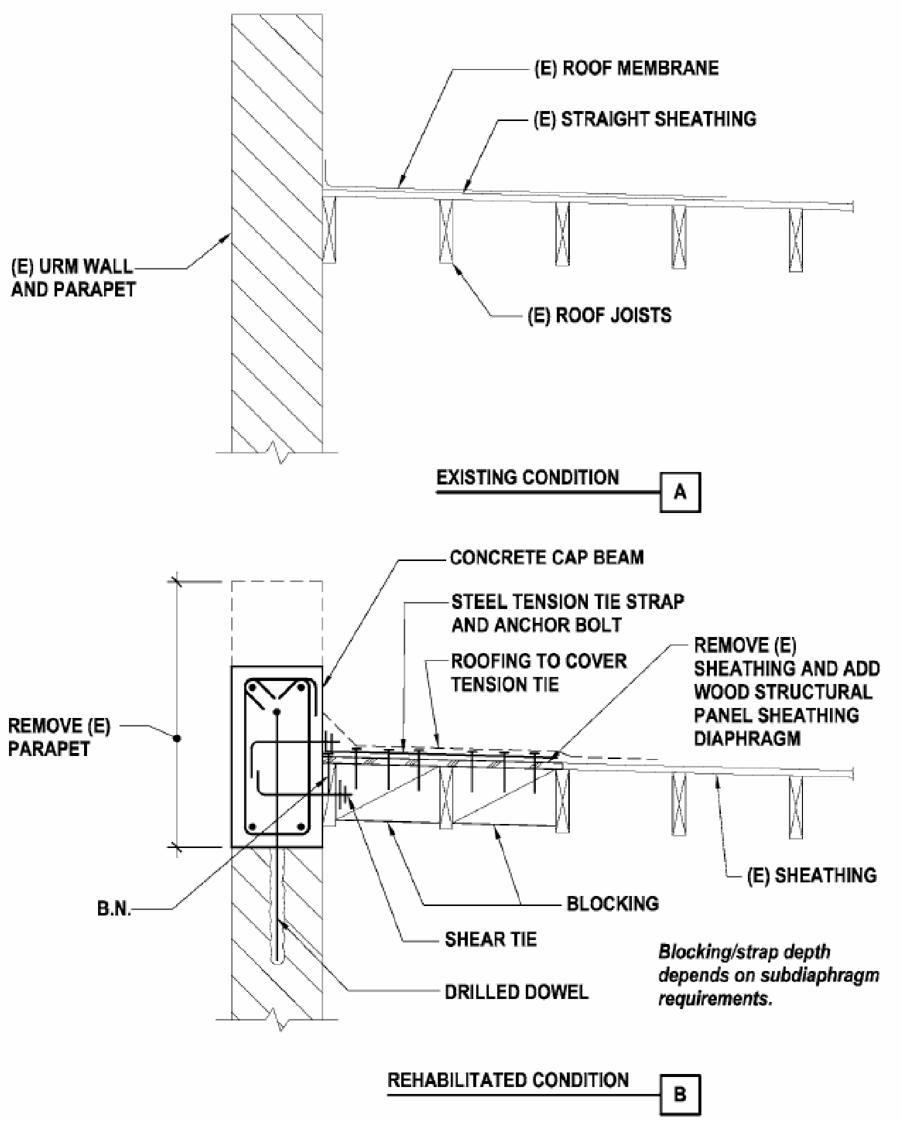


Figure 2.21 Details of parapet removal and installing cap beam in URM buildings, after FEMA 547

2.3.4.2. Enhance existing elements: out-of-plane wall bracing

It is possible to use two types of out-of-plane bracing: 1) diagonal braces that reduce the effective height that is likely to experience sway in the wall (see Figure 2.22a); 2) vertical braces spanning the full height of the inside face of the wall (see Figure 2.22b).

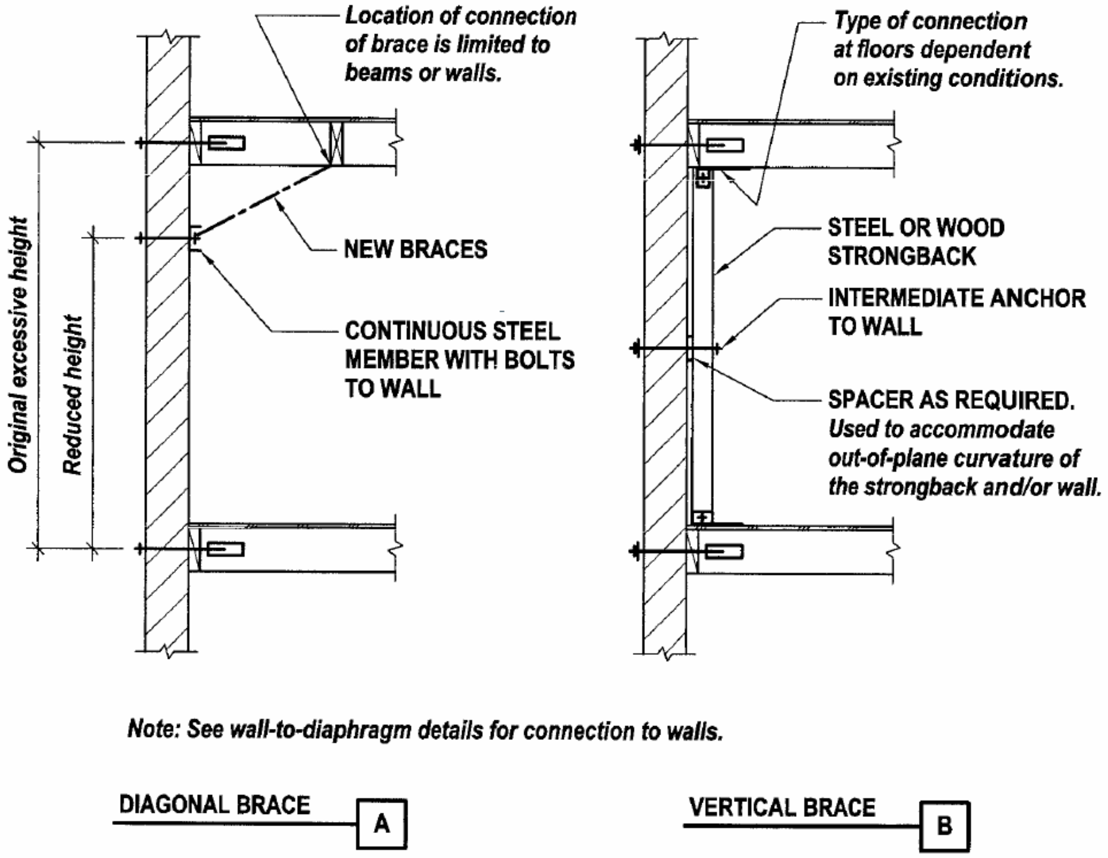


Figure 2.22 Details of diagonal and vertical braces for the walls of URM buildings, after FEMA 547.

Design and Detailing Considerations:

The spacing between vertical braces should not exceed the minimum of 3 m or half of the unsupported (unbraced) wall height. The maximum spacing for diagonal braces is 1.8 m. Typical details of connecting the vertical braces to existing masonry walls via drilled dowels or bolts with anchor plates are reported in Figure 2.23. The braces are generally made of steel, but the vertical ones can be done with strong wood posts or concrete pilasters. Such alternatives are not common in the Palestinian construction industry; therefore, they are now shown here.

Cost/Disruption Considerations:

Although diagonal bracing is less expensive than vertical one, it is considered less reliable. The installation process of any type of bracing is considered disruptive as it must be carried out around the entire building perimeter. It also involves drilling of dowels, accessing and constructing connections with existing diaphragms.

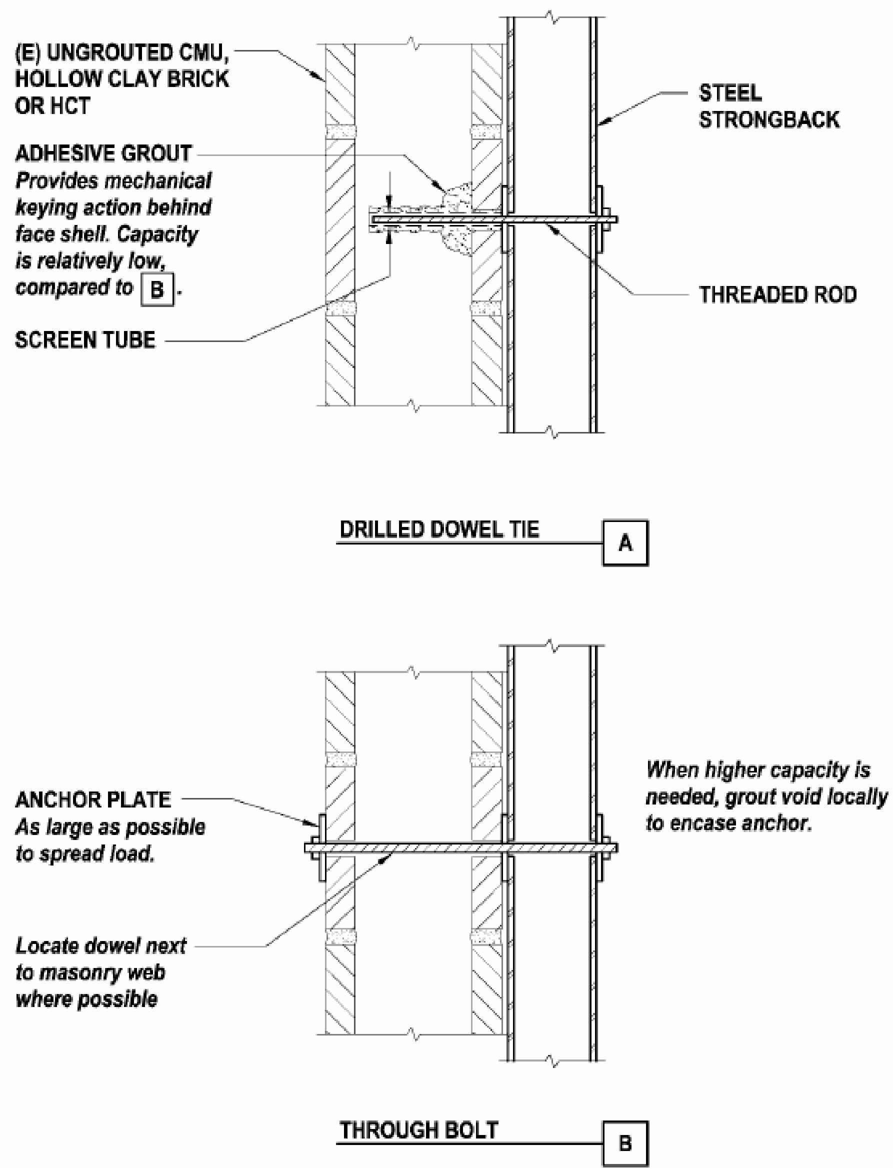


Figure 2.23 Connection between vertical braces and a) drilled dowels; b) through bolt, for URM buildings, after FEMA 547 (FEMA 2006).

2.3.4.3. Enhance existing elements: add reinforced cores to URM walls

This retrofitting technique can be implemented for enhancing both the out-of-plane or in-plane strength of URM walls. This involves drilling a reinforced core from the roof down the inside of an unreinforced masonry wall. A steel reinforcing bar and grout are also placed inside the hole to increase the strength of the wall. The main advantage of such a technique is that it has no adverse impact on the aesthetics of the existing walls, unlike vertical and diagonal bracing systems discussed earlier. A section and elevation views, along with some geometric details, for URM walls strengthened through reinforced cores, are illustrated in Figure 2.24. Moreover, a plan detail for the reinforced core itself is provided in Figure 2.25.

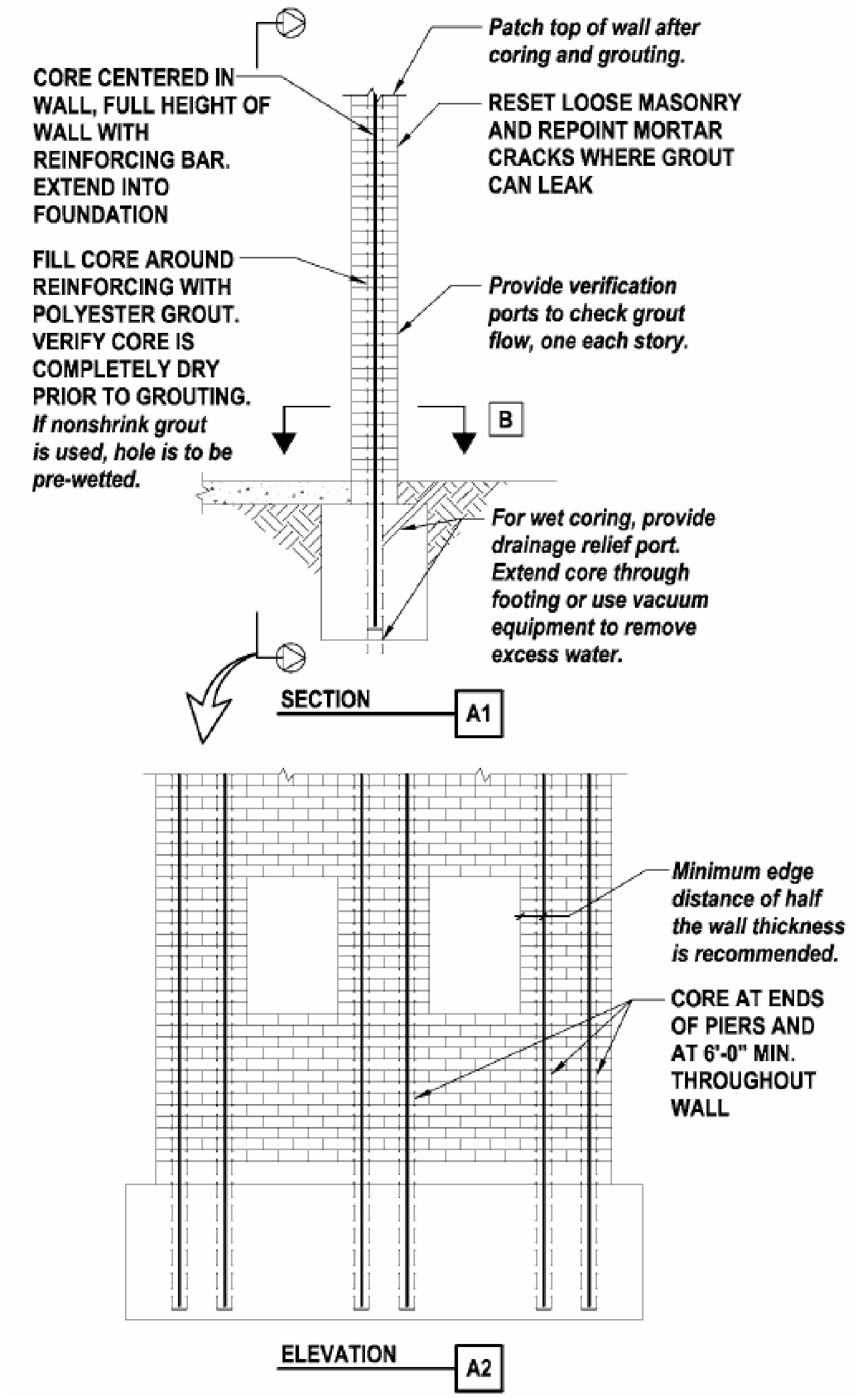


Figure 2.24 Section and elevation views of URM walls strengthened with reinforced cores, after FEMA 547 (FEMA 2006).

<u>Design and Detailing Considerations</u>:

The reinforcing bars used for the reinforced cores are the same as those used to design typical reinforced-concrete elements. The bar diameters are usually between 16 and 25 mm. A minimum spacing between the drilled reinforced cores of 1.8 to 3 m is desirable. The top of the URM wall must be accessible to facilitate the drilling process.

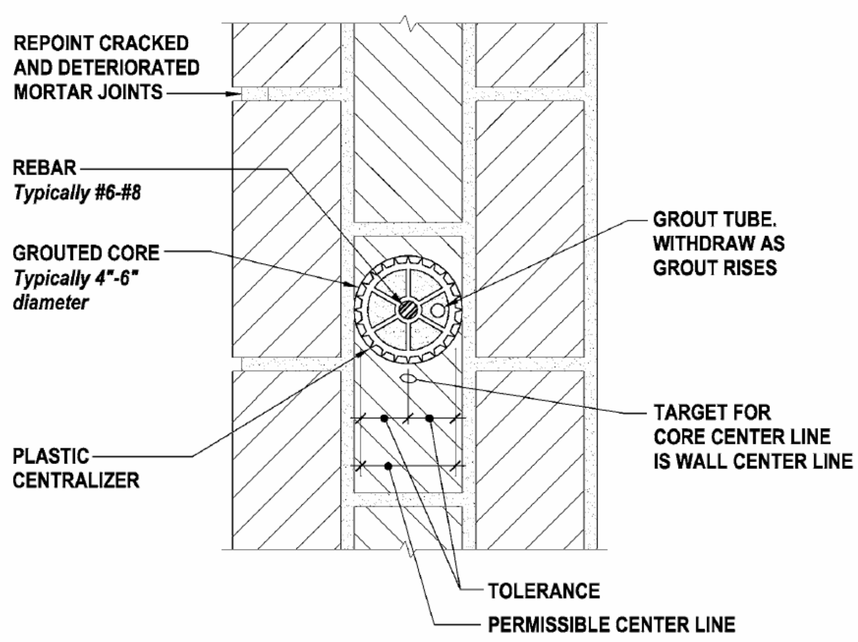


Figure 2.25 Plan detail for a reinforced core used to strengthen URM walls, after FEMA 547.

<u>Cost/Disruption Considerations</u>:

Installing reinforced cores within URM walls could be significantly more expensive compared to traditional diagonal or vertical steel bracing. Therefore, such a technique is typically performed for buildings with historical and architectural value rather than ordinary residential and commercial buildings. The disruption is minimal in the case of reinforced cores as they are drilled within the existing URM walls. Drilling equipment will cause noise and vibration.

2.3.4.4. Enhance existing elements: concrete wall overlays

Adding concrete overlays can improve in-plane strength, but also could be useful for the out-of-plane bending strength. The new concrete layer is attached to the URM wall via adhesive anchors. The thickness of the concrete layer usually varies between 100 and 300 mm. A visual illustration for concrete overlays is provided in Figure 2.26. Due to the significantly higher strength of concrete compared to masonry, it is possible to assume that 100% of the seismic demand is taken by the concrete overlay. Otherwise, the load will be sheared through the relative rigidity of both components (i.e., URM wall and concrete overlay). Such force-based design approaches are much more common in the engineering industry compared to displacement-based ones. It should be noted that when the concrete overlay is added to the URM wall, it is additional inertial load resulting from seismic excitation of the increased weight must be considered in the design and assessment process.

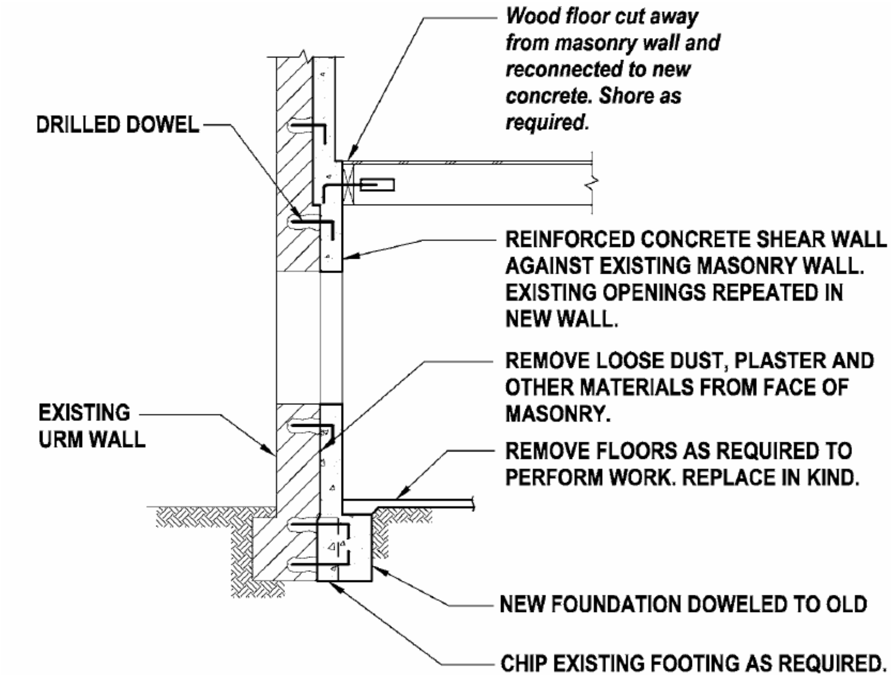


Figure 2.26 Concrete overlays for URM walls, after FEMA 547 (FEMA 2006).

<u>Cost/Disruption Considerations</u>:

Using cast-in-place concrete overlays requires access for the hose and concrete truck. The process of placing concrete is noisy, in addition to the noise caused by labors and concrete pump/trucks. However, cast-in-place concrete is usually a non-expensive technique.

2.3.4.5. Enhance existing elements: FRP overlays

FRP overlays are primarily installed to improve the in-plane URM wall shear strength, but such an overlay can also enhance the out-of-plane flexural strength. The surface of the existing wall must be prepared, and after placing the FRP overlay it must be protected against ultraviolet rays. There is no codified design basis for FRP overlays used to strengthen URM walls. However, additional information can be obtained from manufacturers and other available FRP-related guidelines. Figure 2.27 provides a schematic view of the steps needed to apply FRP overlays to strengthen URM walls.

<u>Design, Construction, and Detailing Considerations</u>:

The surface of existing URM wall must be cleaned from loos material and finishes that prevent sufficient adhesion to the FRP overlays. FRP strips are placed horizontally to improve the in-plane shear strength. They are placed vertically if out-of-plane bending strength enhancement is needed. On the other hand, diagonal strips can be adopted to resist diagonal tensile stresses from in-plane shear.

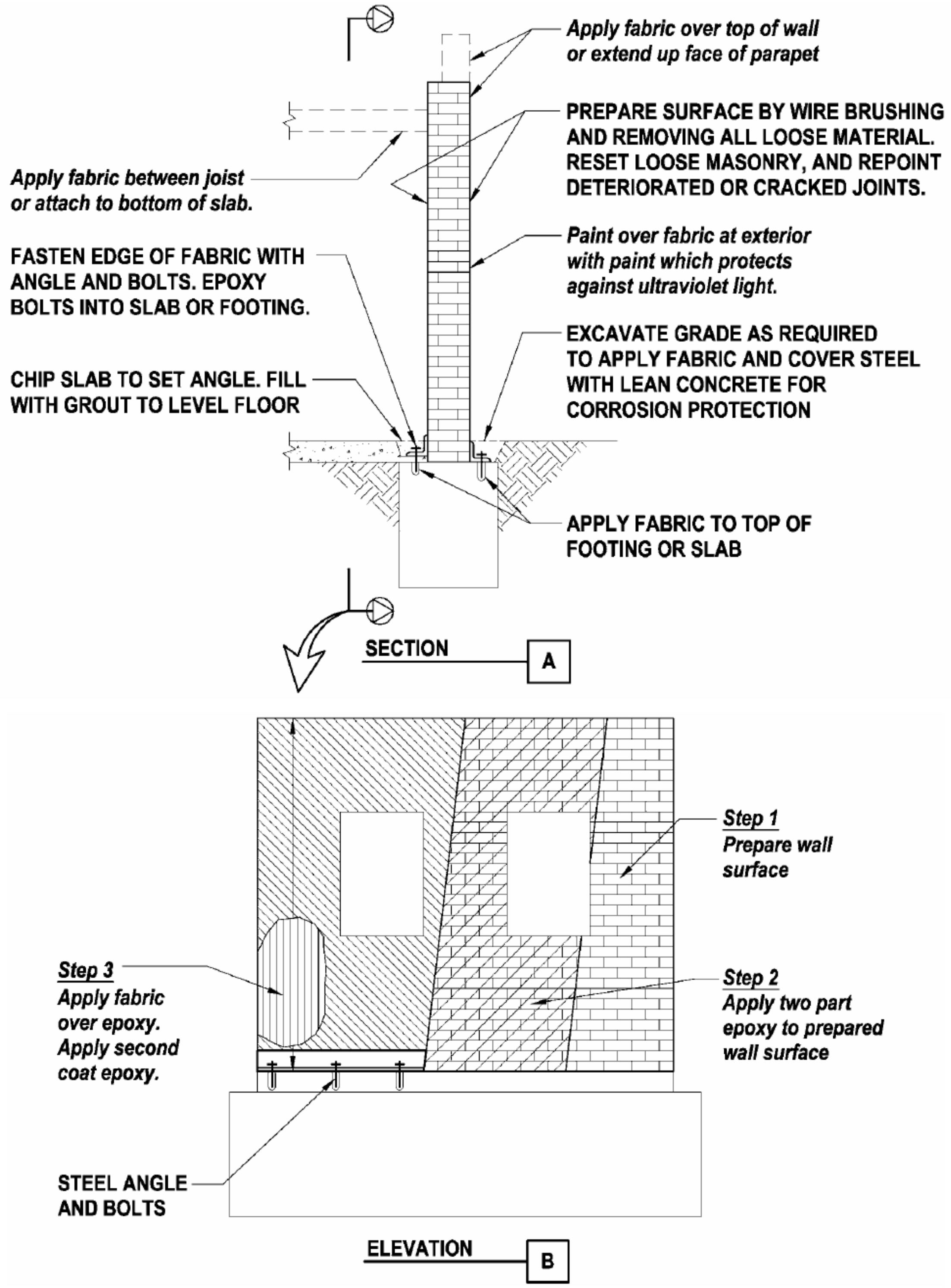


Figure 2.27 Details on FRP overlays for strengthening URM walls, , after FEMA 547 (FEMA 2006).

Cost/Disruption Considerations:

The use of FRP overlays for strengthening URM walls is not very common in practice due to the relatively high cost. Minimum disruption and impact to the building aesthetics are normally involved in the application of such a retrofitting technique.

2.3.5. Rehabilitation of Foundations

The techniques and procedures adopted in the rehabilitation of foundations are usually common among multiple building types (e.g., C1, C2, C1a, URM). Although adding new foundations is a standard procedure on many occasions when adding new structural elements is involved in the rehabilitation strategy, the strengthening of existing ones is quite less common. This is because foundation work in existing structures is generally very expensive. Also, foundation analysis is one of the most challenging areas of seismic retrofitting and rehabilitation. Various assumptions regarding support conditions, soil properties and location could significantly affect the results. When detailed analysis reveals that new foundations must be added or existing ones must be retrofitted, the design engineer must have a good understanding of soil-related and geotechnical issues. Many details on the type of testing and modeling approaches that can be used for foundations are available in FEMA 547 (FEMA 2006). The current guidelines provide solely some engineering recommendations and brief information about the potential foundation retrofitting techniques as summarized in the following sections.

2.3.5.1. Add a shallow foundation next to existing shallow foundation

New shallow foundations must be added next to existing ones in multiple cases. For instance, when a concrete overlay is used to retrofit an existing wall, a new footing is typically needed. Figure 2.28 reports a schematic illustration for a new concrete strip footing added next to existing ones upon retrofitting the existing wall with a concrete overlay.

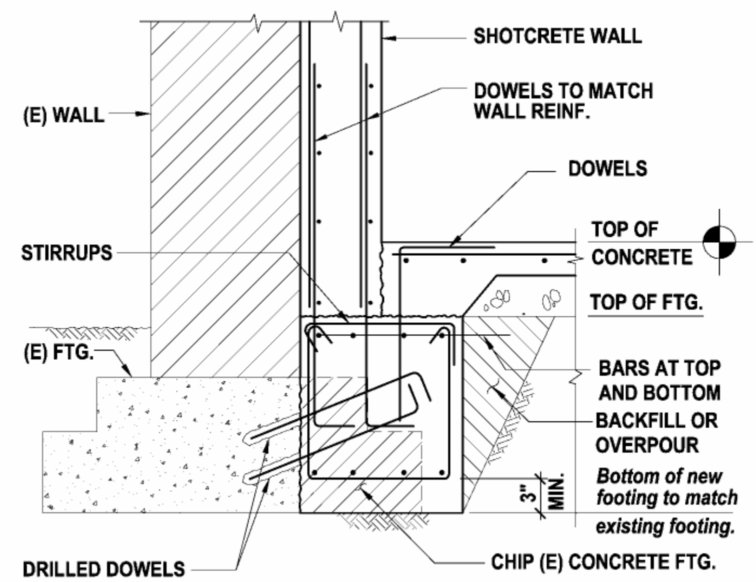


Figure 2.28 New concrete strip footing next to an existing one, after FEMA 547 (FEMA 2006).

It can be noticed in Figure 2.28 that the existing and new footings are connected through drilled dowels to ensure an appropriate shear transfer. For design simplicity, it is possible to assume that the new footing resists the loads associated with the added overlay. Otherwise, the loads could be shared between the new and existing footing on the basis of the area. A more complex, yet more accurate analysis requires considering the relative stiffness of the soil under the existing footing and that under the new one.

2.3.5.2. Enlarge an existing footing

Footings (e.g., isolated, mat, combined) might be subjected to tensile or compressive stresses that exceed their capacity. Therefore, it is possible to enlarge existing footings to increase their compression strength, or to increase dead loads that resist tensile stresses. However, achieving large increases in compression strength is usually difficult due to the limits of the existing footing. An example is shown in Figure 2.29. Reinforcing bars must be drilled on the sides of the existing footing, but without penetrating towards the opposite side. The bending strength must be checked at locations "A" and "B". Location "A" typically governs the design process.

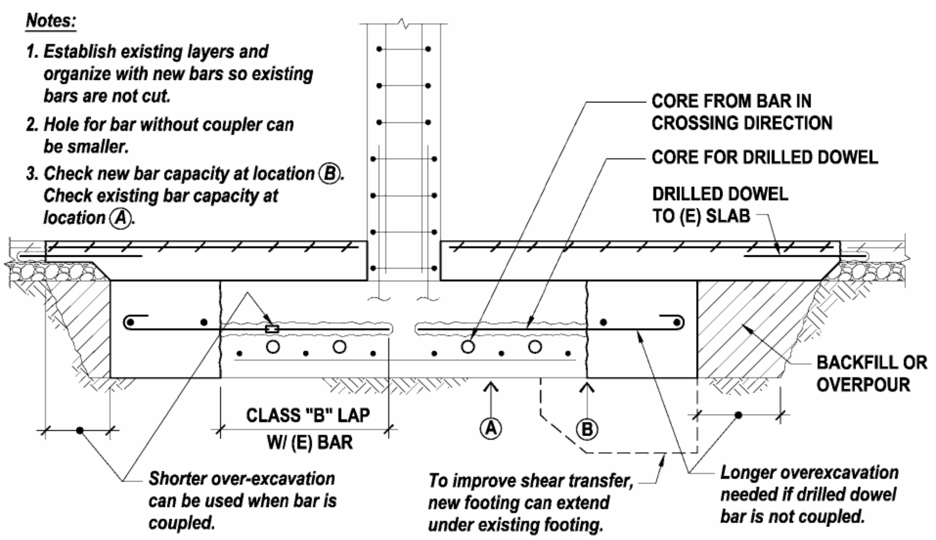


Figure 2.29 Enlargement of existing footings, after FEMA 547 (FEMA 2006).

ملخص بالعربية

يهدف هذا العمل لوضع إرشادات حول الآليات المنهجية التي يمكن استخدامها لتقييم الأداء الإنشائي للمباني القائمة في حال حدوث هزة أرضية، و ذلك من خلال تقنيات النمذجة و التحليل الإنشائي المتطورة. كما تم وضع معايير يمكن من خلالها قياس مدى توافق الأداء الإنشائي للمباني القائمة مع متطلبات الأداء الزلزالي المقترحة في الكودات الهندسية، بما يحقق الدرجة المقبولة من الأمان. و تميز هذه المعايير بشكل كبير بين المباني بالاعتماد أهمية المبنى و دوره في قدرة المجتمع على الصمود أمام الكوارث الطبيعية. ولهذا، تكون المعايير صارمة جداً في حالة مباني البنية التحتية مثل المدارس والمستشفيات ومراكز الدفاع المدني لأن مثل هذه المنشآت يجب أن تعمل كالمعتاد في أثناء و ما بعد الكوارث الطبيعية، بينما تكون المعايير المطلوبة في المباني العادية مثل السكنية و التجارية الصغيرة أقل صرامةً حيث أنها تركز على منع الانهيار. تتوافق جميع هذه المقترحات مع الكودات و المنهجيات العالمية، خاصة تلك المستخدمة في الولايات المتحدة الأمريكية و دول الاتحاد الأوروبي.

تم أيضاً وضع استراتيجيات تصميمية و تفصيلية لتقوية و تدعيم الأبنية القائمة الدارجة في فلسطين، و التي يتبين أنها لا تتوافق مع متطلبات الأداء الإنشائي ضد الكوارث الطبيعية بعد تنفيذ منهجية التقييم الإنشائي المقترحة في هذا التقرير. تم تصنيف هذا الاستراتيجيات بناءاً على نوع المشاكل الإنشائية من جهة، مثل نقص القوة، نقص الصلابة، خلل في الشكل المعماري، خلل في تفاصيل العناصر الإنشائية، و غيرها. كما تم تصنيف الاستراتيجيات من جهة أخرى بالاعتماد على طبيعة التدخل المطلوب مثل بناء عناصر إنشائية جديدة، تدعيم عناصر إنشائية قائمة، تقوية الترابط بين العناصر الإنشائية، إزالة عناصر إنشائية ذات خطورة عالية، و غيرها. بناءاً على هذا، تم عمل جداول مبسطة تتضمن الحلول التدعيمية المقترحة بناءاً على نوع المشكلة و طبيعة التدخل لجميع أنواع المباني الدارجة في فلسطين. كما تم أخيراً مناقشة بعض التقنيات الوارد الممكن استخدامها بالسوق الهندسي الفلسطيني من ناحية الميزات، العيوب، آلية التنفيذ مع مخططات و صور توضحية، التكلفة، و التحديات التي قد تواجه عملية البناء و التنفيذ في الموقع. كما و تم الإشارة إلى جميع الكودات العالمية التي يمكن الرجوع إليها من قبل المهندسين و أصحاب الاختصاص للحصول على التفاصيل الكاملة حول آلية التقييم و التدعيم للأبنية القائمة.

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