



Nepal Health Sector Support Programme III (NHSSP – III)

Nepal Earthquake Retrofitting & Rehabilitation Standards

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SYNOPSIS

1.1 Scope of document

Given the seismicity of Nepal and the fact that many of the healthcare buildings are older and non-compliant to modern seismic codes, the next major earthquake could have catastrophic consequences. This fact can serve as a warning and as motivation to undertake a comprehensive seismic assessment and retrofit program for healthcare facilities. This standard has been prepared to address this issue.

In other developing countries located in seismically active regions, similar documents have been developed and used to retrofit critical buildings. An example is the development of the retrofitting standard and practice for Istanbul (Turkey) public buildings. The World-Bank sponsored multiyear project has resulted in the retrofit of more than 1,000 school and hospital buildings. It is anticipated that the human and physical cost are significantly reduced as the result of implementation of such programs. Projects of similar scope are currently under consideration or implementation in Asian countries such as the Philippines and Indonesia.

This standard has been developed to address the seismic deficiencies of healthcare facilities (hospital buildings) in Nepal. The standard has been developed to help engineers develop seismic safety retrofits that would enhance the seismic safety of individual buildings and, when applied properly, would improve the seismic resiliency of the overall healthcare community.

This document is intended for use as a supplement to the national building code of Nepal: Nepal National Building Code, NBC 105: Seismic design of Buildings in Nepal and the Indian Standard IS 1893: Criteria for earthquake resistant design of structures. NBC105 is the legal technical seismic design code in Nepal for the design of new structures and IS1893 is the commonly used seismic Standard in Nepal. Participating engineers should be intimately familiar with its specifications. For reference, key provision of the codes and other pertinent documents are summarized at the end of this standard.

The NBC105 is currently under revision. It is understood that NBC105 will be substantially changed. Once updated NBC105 is promulgated as the accepted Standard, this document will also require necessary amendments to harmonize it with the updated NBC105.

The key difference between this standard and the national code is that by definition less conservatism is implied in the retrofit of existing buildings due to several contributing factors. Among them, i) the existing buildings have shorter design life than the new buildings with an expected useful life of 50 to 75 years; ii) it is important to encourage the seismic retrofitting of as many buildings as possible within a given financial constraint. The lower threshold for conservatism does not imply less safe buildings, as the structures retrofitted using the provisions of this standard are expected to perform well in earthquakes.

1.2 Reference document

This standard relies on the provisions of FEMA 356 ([FEMA 2000](#)) in prescribing seismic assessment and retrofit methods. FEMA 356 is an open-sourced document and is the culmination of a collaborative effort by the Federal Emergency Management Agency (FEMA) and the American Society of Civil Engineers (ASCE) in the United States. It provides a methodical and rational approach to seismic assessment and retrofit, incorporating state-of-the-art research, findings from major earthquakes, and engineering judgment. The choice of FEMA 356 instead of the more recent editions was based on the age and construction methods used on buildings in Nepal and because this document is readily available online at no charge for local engineers to use as reference.

1.3 Choice of retrofit solution

This standard provides a number of retrofitting solutions. The engineers are to assess the existing condition of a building, identify its deficiencies, and then select the seismic retrofit solution that is the most suitable.

In many instances, in particular for the existing concrete frame buildings, the option of adding new concrete shear walls (RCSWs) could prove to be the optimal retrofit solution for buildings that have inadequate lateral stiffness and strength. Well designed and constructed RCSW buildings have performed very well in past earthquakes. In addition, RCSW provide the following advantages for seismic retrofitting of reinforced concrete framed buildings in Nepal: i) can be designed to carry 100% of earthquake loading, thereby reducing analytical work, ii) add stiffness to the building, reducing deformation (drift) and thereby reducing damage to masonry infill walls iii) could be economical and can be built using the material available locally, iv) are simple to construct and thus, local contractors can easily build them, and v) walls can be incorporated into the existing bays of the building framing or bearing walls. Accordingly, procedures for RCSW for the reinforced concrete buildings are presented as one of the options for retrofitting, however, during the investigation stages, other options should also be evaluated carefully.

For Nepalese unreinforced masonry buildings, which usually have sufficient stiffness but mostly lack integrity, strength and ductility needs to be looked at differently. In this context, the goal of the intervention would be to improve ductility and strength. Hence, the various options need to be investigated to correct the identified deficiencies.

However, the selection of seismic retrofitting for a particular building is highly dependent on the results of the seismic assessment and the seismic deficiencies. Therefore, it is recommended that the engineer carefully examine the deficiencies and quantify them, and select the retrofit solution that best addresses these deficiencies.

1.4 Application methodology

It is recommended that practicing engineers follow the flowchart in Figure 1 when they apply the provisions of the standard.

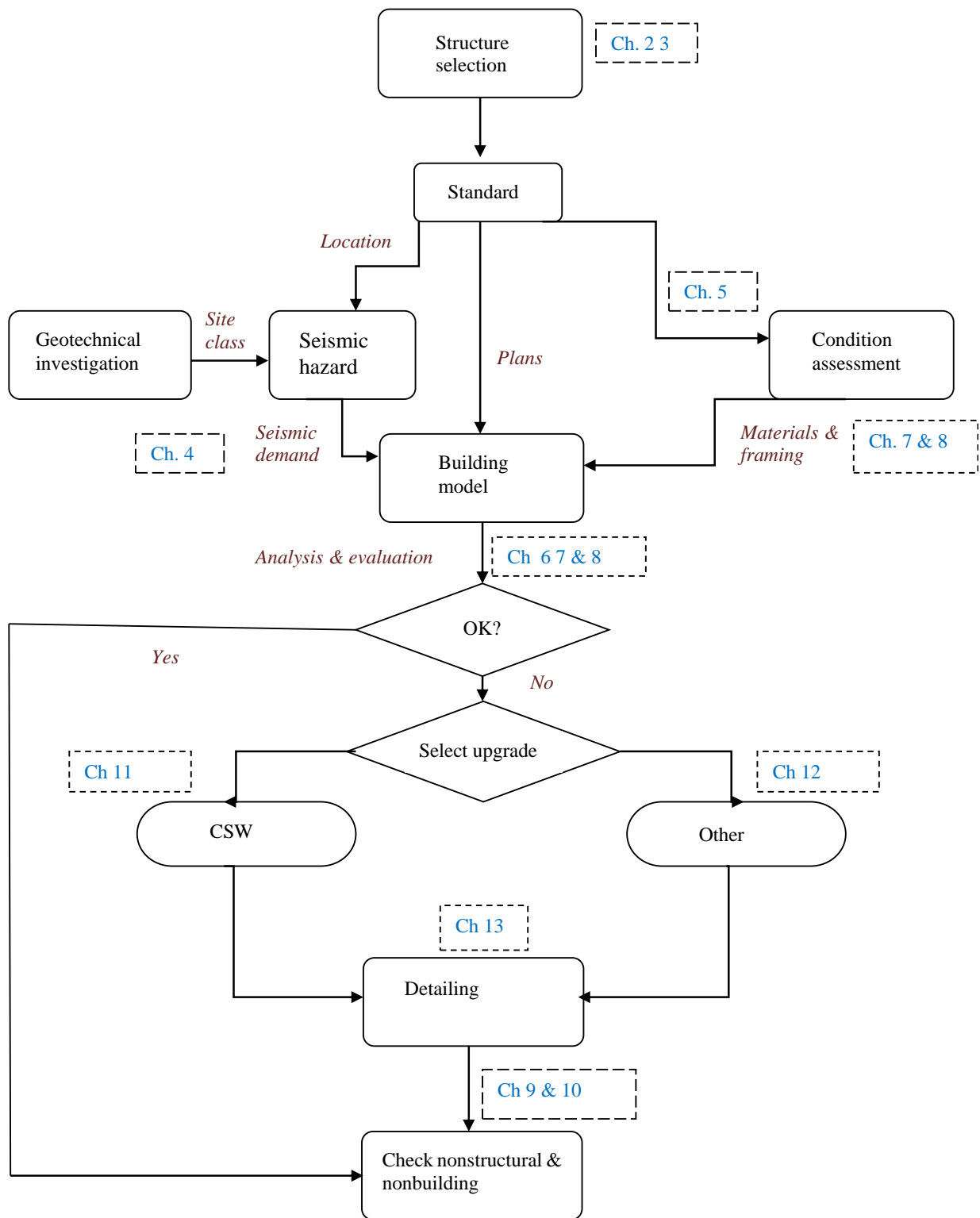


Figure 1. Flowchart for application of the standard

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ACRONYMS, ABBREVIATIONS, AND NOTATIONS

The definitions for equation variables that are used in this document are given with the equations. In addition, the following acronyms, abbreviations, and notations are used throughout the document.

Technical Organization Acronyms

ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
FEMA	Federal Emergency Management Agency (U.S.)
IS	Indian Standard
NBC	National Nepal Building Code

Technical Acronyms

DE	Design earthquake
GFRS	Gravity-force-resisting system
LFRS	Lateral-force-resisting system
LS	Life Safety
LSP	Linear Static Procedure
<i>m</i> -factor	acceptable demand to capacity ratio
RC	Reinforced concrete
RCSW	Reinforced concrete shear wall
RSP	Response Spectrum Procedure

Notations

δ	(story) drift
g	Acceleration of gravity (9.81 m/s^2)
h	(building) height
I	Importance factor
κ	Knowledge factor
PGA	Peak Ground Acceleration
Q	Action (force or moment) acting on a component
R	Response reduction factor
S_a	Spectral acceleration
T	Building period (seconds)
t	Thickness
V	(Base) shear
W	Weight

2. INTRODUCTION

2.1 Overview

The *Seismic retrofitting and rehabilitation standards for health infrastructures in Nepal* (hereinafter referred to as “Standard”) have been developed to assist in addressing the seismic retrofitting design requirements for existing hospital buildings in Nepal. This document is recommended for use as a supplement to the most recent Nepal Building Code.

2.2 Design Basis

The Standard specifies recommended procedures for the seismic assessment and retrofit of healthcare buildings. Seismic assessment is defined as a process or methodology for evaluating the deficiencies in a building. Seismic retrofit is defined as the process of improving the seismic performance of a building by correcting the deficiencies identified in a seismic evaluation.

The seismic assessment procedure shall be based on the as-built information and/or a site visit, including:

- General building description (number of stories and dimensions)
- Structural system description including framing, lateral-force-resisting system (LFRS), floor and roof diaphragm construction, basement, and foundation system
- Hospital building type
- Material properties and site conditions
- List of identified seismic deficiencies

Seismic retrofit of an existing building shall be achieved by implementing retrofit measures that address the deficiencies that were identified by the seismic evaluation. The effects of the retrofit on stiffness, strength and deformability shall be taken into account in the analytical model of the retrofitted structure. The compatibility of new and existing components shall be checked. One or more of the following strategies are permitted as retrofit measures:

- Add new structural elements
- Improve detailing for the transfer of lateral forces from horizontal (floors) to vertical (walls or columns) elements and to foundation (load path)
- Improve the connectivity and diaphragm action at floors and out-of-plane resistance

The Standard *implies* the following performance objective for a given level of seismic intensity as follows:

- The building is expected to preserve life safety (LS) and not collapse during the design earthquake. This is the implied level of performance in the modern seismic codes.

This is consistent with the expected level of performance of retrofitted structures with seismic resilience of other similar new structures designed following the relevant seismic standards.

The following assessment, retrofit and maintenance steps are recommended:

- The structural system should be clearly defined, and properties that are specific to different systems, such as walls and frames, should be identified.
- A site investigation should be performed to assess the condition of the as-built structure. The analytical model of the building should accurately represent the physical structure.

- The proper design of members and their connections to one another and a continuous and redundant load path is crucial for satisfactory seismic performance.
- The retrofit configuration should be simple and regular and should meet aesthetic requirements.
- The retrofit addresses the issues such as minimum intervention, least cost, minimum downtime options.
- Good quality control is necessary to help ensure that the retrofit is properly constructed.
- A regular and thorough maintenance program is required to help ensure that the building retains its integrity over time. Corrosion of steel, concrete cracking and spalling, and foundation integrity (e.g., settlement) should be remediated where encountered.

2.3 Scope and Limitations

This document is intended for use for typical building types as described in the following sections. The intent of the document is to address the majority of health structures in Nepal. To encourage seismic retrofitting, prescriptive measures are provided that have been used successfully elsewhere.

However, when site-specific considerations would classify the building outside of this scope, a more detailed investigation and analysis is recommended, and the provisions of this standard might not be appropriate. Such cases include:

- For hospital buildings with complex geometry, highly irregular, or mixed construction that do not lend themselves to the analysis procedure desired hereafter
- Buildings considered critical facilities, and for which a higher level of performance (immediate use and functionality) than described in this document is required
- Site conditions subjected to ground shaking intensity for which site-specific hazard must be developed
- Site conditions where significant liquefaction, lateral spreading or ground settlement are present

2.4 Applicability

The seismic assessment and retrofitting of a facility (i.e., building or a group of buildings, including non-building structures, etc.) shall evaluate the seismic risk and its mitigation holistically. It shall address all the elements of the facility including principal structures (building or non-building such as canopies, walkways, access ways, water tanks, etc.), other elements (e.g., facades, parapets, gables, etc.) and non-structural elements (e.g., false ceiling, mechanical and electrical services, etc.) which could cause life-safety hazard and/ or disruption of function of a facility.

2.5 Organization of the Standard

The Standard provides a prescriptive methodology for evaluating and upgrading hospital buildings. The approach used in the Standard is based on the following:

- Apply the Standard to assess a building in its current configuration.
- If the building is inadequate, use the retrofit options stated in the Standard.

Following is a summary of the basic steps:

- Select an applicable structural system.
- Determine the seismic hazard from the seismic standard.

- Review geotechnical hazards at the site.
- Perform a condition assessment, including materials testing, and establish the knowledge factor, κ .
- Assess the performance of the building qualitatively based on on-site investigations, review of documents and observation of damage to the similar buildings in the past earthquakes.
- Prepare a mathematical model (hand calculations and computer model) of the building.
- Perform linear static analysis by using the procedures in the Standard.
- Assess the performance of the building quantitatively. Verify qualitative assessment.
- If performance is inadequate, select retrofit solution.
- Design new retrofit components or improve the existing components with:
 - Strength to carry 100% of the lateral load
 - Drift ratio limited
 - Detailing as provided in the Standard
- Check secondary structural components anchorage and bracing a retrofit as necessary.
- Check nonstructural components anchorage and bracing a retrofit as necessary.
- Check and retrofit non-building structures.

3. BUILDING TYPES

3.1 Overview

In general, the healthcare infrastructure in Nepal can be divided into two broad groups: i) the larger urban hospital buildings and ii) the rural health facilities with a small footprint. For the former, the construction uses reinforced concrete framing or unreinforced masonry, whereas, the latter is mostly unreinforced brick or stone masonry construction. The building types can be further grouped based on the mortar used in construction as listed in Table 1.

Type	System	Description			Stories	Comments
		Lateral system	Floor	Roof		
B1		Irregular shaped stone in mud mortar	Timber/ concrete	Light gage metal or slate roof on steel or timber	1 or 2	Most vulnerable
B2	Unreinforced masonry bearing wall	Brick with mud mortar	Concrete or timber	Light gage metal or slate roof on steel or timber	1 or 2	Better performing than B1
B3		Regular (semi dressed or dressed) stone with/ without mud mortar	Timber	Light gage metal or slate roof on steel or timber	1 or 2	
B4		Brick with cement mortar	Concrete	Concrete or light gage metal or slate roof on steel or timber	1 to 3	Better performing than B2 and B3
B5		Irregular/ regular shaped stone with cement mortar	Concrete	Concrete or light gage metal or slate roof on steel or timber	1 to 2	
B6	Moment frame	Cast-in-place reinforced concrete (RC) moment frame with unreinforced brick or masonry infills	Concrete	Concrete	1 to 5	Non-ductile, seismic non-compliant
B7			Concrete	Concrete	No limit	Generally, <20 years old, ductile, seismic compliant
B8	Bearing wall	Mixed: stone and brick in mud, brick in mud and cement	--	Light gage metal, wood, or truss	1	Not considered explicitly, assess and retrofit per B1-B5
B9	Moment frame	RC moment frame without infills	Concrete	Concrete	--	Not considered in this document due to small number of buildings
B10	Shearwall	Cast-in-place reinforced concrete moment frame	Concrete	Concrete	--	
B11	Reinforced masonry bearing wall	Reinforced concrete block masonry	--	Light gage metal or steel roof	--	

Table 1. Building typologies

3.2 Seismic vulnerability

Among the building types considered, Type B1 is expected to be the worst performing, whereas, Type B7 that meets requirements of the building code would perform the best and likely require no major structural retrofitting. A large number of building construction in Nepal (B1-B6 and B8) can be mainly classified as nonductile concrete frame, or unreinforced masonry bearing wall construction. These types are the most vulnerable to collapse or severe damage during earthquakes. Accordingly, it is essential that careful assessment of the buildings be conducted to identify seismic deficiencies and then seismic retrofitting be undertaken to address such deficiencies.

3.3 Seismic deficiencies

The factors contributing to the seismic vulnerability of the stated building types are summarized in this section.

3.3.1 Concrete frame buildings with masonry infill

- Building irregularity* as a result of nonstructural partition walls inadvertently resisting seismic loading
- Captive columns due to stairways or partial height infill masonry
- Pounding of adjacent buildings due to the lack of seismic gap
- Out of plane failure of infill walls due to lack of anchorage between these walls and floor slabs
- In-plane failure of infill walls as they attract seismic force due to high stiffness of these walls compared to the concrete framing. These walls also could lead to shear failure of columns
- Low strength concrete and poor construction
- Lack of ductile detailing for concrete members including: inadequate lap splice, lack of confinement, use of stirrups at large spacing or without 135-degree hooks, inadequate embedment of slab reinforcement to the concrete columns, lack of beam column joint ties; inadequate embedment of beam reinforcement into the columns
- Lack of expansion joint between the stairways and the slabs
- Unbraced parapets, gables, partitions
- Deterioration as a result of poor maintenance

Many of the RC frame buildings with masonry walls may not appear to have a soft or weak story. However, once the infill walls of the lower story crack during seismic shaking, it could potentially create soft/ weak-story conditions.

3.3.2 Bearing wall (brick or stone) buildings

- Use of mud mortar or weak cement mortar
- Use of irregular or round stones for construction of stone masonry walls
- High volume of mud mortar
- Poorly integrated multi-leaf stone walls and potential delamination
- Lack of out-of-plane anchorage

* This is in addition to structural irregularities (e.g. captive column, soft story, plan and vertical irregularity) than must be assessed and retrofitted.

- In-plane shear capacity of walls
- A lack of good connection between return walls
- Lack of connection and/or deficient connections between diaphragms and the walls
- Flexible diaphragm not providing any meaningful in-plane capacity to hold the walls
- Inadequate capacity of flexible diaphragms
- Unbraced parapets, gables, partitions
- Deterioration of mortar and other elements as a result of poor maintenance

3.3.3 Alterations

In many instances, additional floors have been added to existing buildings. This presents the following issues:

- New structure uses different structural system than the existing building
- New structures use masonry walls not properly tied to the existing system and thus would act as cantilevers
- Additional weight (seismic mass) of these walls was not included in the design phase

3.3.4 Nonstructural components

The two key elements to consider for nonstructural components are the adequate bracing and anchorage. For heavy equipment, in many cases, proper anchorage to the walls or floors is not provided. For ducts, piping and other distributed systems, adequate bracing is typically not provided.

4. SEISMIC DESIGN PARAMETERS

4.1 General

This chapter provides general recommendations on seismic design loading for seismic assessment and retrofitting design of existing buildings and expected seismic performance requirements. The seismic design standard of Nepal (NBC105) was prepared in 1994. The NBC105 is in the old form - it provides design spectra for a ductile moment frame and uses the structural performance factor, K , while the current worldwide trend is to drop the performance factor K and replace it by reciprocal of R , response reduction factor or similar to reflect the building's structural system and available ductility.

The Department of Urban Development and Building Construction (DUDBC) has initiated a process for updating NBC105. Hence, in the interim, Indian seismic standard, IS1893:2016 has been proposed for use for estimation of seismic forces for seismic assessment and retrofitting design of hospital facilities in Nepal.

Once updated NBC105 is promulgated as the accepted Standard, this document will also require necessary amendments to harmonize it with updated NBC105. Hazards under consideration.

Given the location of Nepal in the middle of the Himalayas, the whole of Nepal is prone to seismic shaking. Depending upon the area, the hospital sites in Nepal are also susceptible to liquefaction, lateral ground spreading, ground settlement, landslide and rockslide during an earthquake. While this document only addresses the hazard associated with building shaking, the engineer responsible for assessment and retrofitting of hospital facilities shall consult a geotechnical engineer familiar with the site if any other geotechnical hazards are present.

4.2 Provisions of National Building Code

National Building Code is the main document that sets minimum provisions for structural safety of building structures in Nepal. Seismic Design of Buildings in Nepal (NBC105:1994) provides provisions for seismic design loading and earthquake resistant construction for building in Nepal. The NBC105 required this standard be used in conjunctions with IS4326-1976 Code of Practice for earthquake resistant design and construction.

The present NBC 105:1994 describes two methods for seismic actions a) Seismic Coefficient method (also known as Equivalent Static Method), and b) Modal Response Spectrum method. The bulk of seismic resistant buildings are designed using equivalent static lateral forces to represent the effects of ground motion due to earthquake on buildings. It is from the assumption that equivalent static forces can be used to represent the effects of an earthquake by producing the same structural displacements as the peak earthquake displacement response. The application of this procedure is limited to reasonably regular structures with limited height. For high-rise (more than 7 stories) and structures with vertical or plan irregularities, modal response spectrum procedure (MRSP) shall be used.

The next, most commonly used seismic loading Standard in Nepal is Indian Standard Criteria for Earthquake Resistant Design of Structures: IS1893:2002 (Part I: General Provisions and Buildings), which has recently been revised as IS1893:2016 (Part I). The IS1893-1984 included Kathmandu in its body and defined it as seismic zone V (highest seismic zone of IS1893). If the Kathmandu Valley is considered Zone V as per IS1893-1984 and provisions of IS1896-2016 are followed, the reinforced concrete buildings and unreinforced masonry buildings in Kathmandu have to be designed or assessed for higher seismic force than recommended by the NBC105-1994.

The IS1893:2016 (Part I) is more comprehensive and elaborate than NBC105-1994. It has a provision to deal with different kinds of building structures (including buildings of higher importance), with some exceptions, such as industrial and stack-like structures.

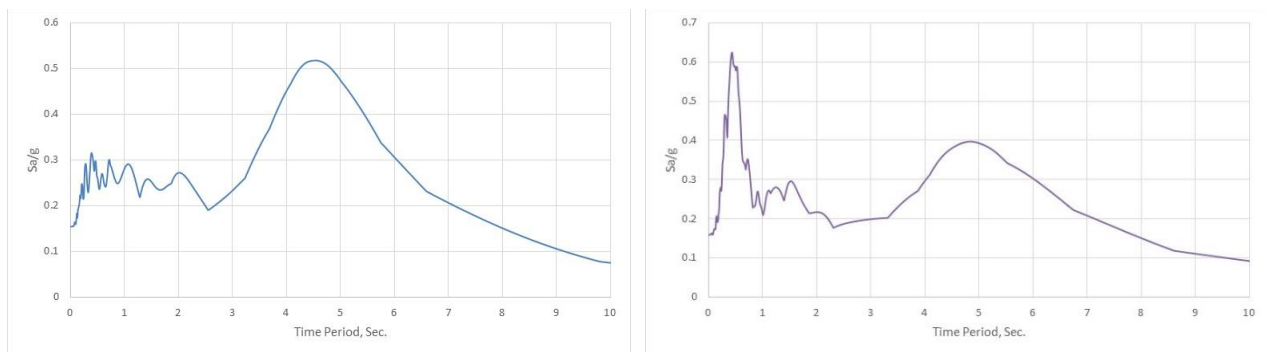
4.3 Recommended accelerations

As discussed earlier, in the interim, IS1893:2016 has been recommended for the purpose of seismic assessment and retrofitting of hospital facilities in Nepal. The whole of Nepal is recommended to be considered Zone V (Seismic Zone Factor, $Z = 0.36$) as per IS1893-2016. Zone V is considered equivalent to intensity IX shaking in 1964 Medvedev–Sponheuer–Karnik (MSK) seismic Intensity Scale.

It should be noted that very different approaches have been adopted for preparation of the 1994 seismic zoning map of Nepal (used in NBC105-1994) and Indian seismic zoning map. The Nepal zoning map is based on probabilistic seismic hazard assessment (PSHA) whereas the Indian zoning map is based on past earthquakes and deterministic seismic hazard assessment (DSHA).

4.4 2015 earthquake seismic parameters

The Response Spectrum functions for the April 25, 2015 Nepal earthquake are shown in Figure 2 for the two components (E-W and N-S) direction earthquake ground motions. These ground motions were recorded on Kantipath station in Kathmandu on soft soil. However, these spectra would not be representative for assessment and retrofitting design of the buildings in Kathmandu as it was recorded 78km away from the epicenter.



Component 1 (E-W)
Component 2 (N-S)
Figure 2. Horizontal components of 25 April 2015 Nepal Earthquake

4.5 Site condition

Site condition should be examined and soil class be determined by site-survey, geo-physical or geotechnical investigations and be classified as Type -I (Hard soil), Type-II (Medium soil), or TYP-III (Soft soil) as per IS1893-2016. The default type Type-III soft soil be considered in analysis if site-specific geotechnical information is not available.

The site-specific geotechnical investigation shall be completed for the areas susceptible to instability, collapsible or liquefiable soils that may cause excessive ground settlements.

4.6 Response spectrum and static procedure

The response spectra curve of Indian Seismic Standards are presented in Figure 3 and Figure 4. The Response Spectrum of Figure 3 shall be used if the building is analysed with linear static procedure (LSP) and Figure 4 is applicable for Modal Response Spectrum procedure (RSP) of analysis.

Response Spectrum are given in IS codes for three types of soils:

- Type I: Rock or Hard soil

- Type II: Medium soil
- Type III Soft soil

The detail description about the classification of the soil type is in IS 1893:2016 (Part I), Clause 6.3.5.2.

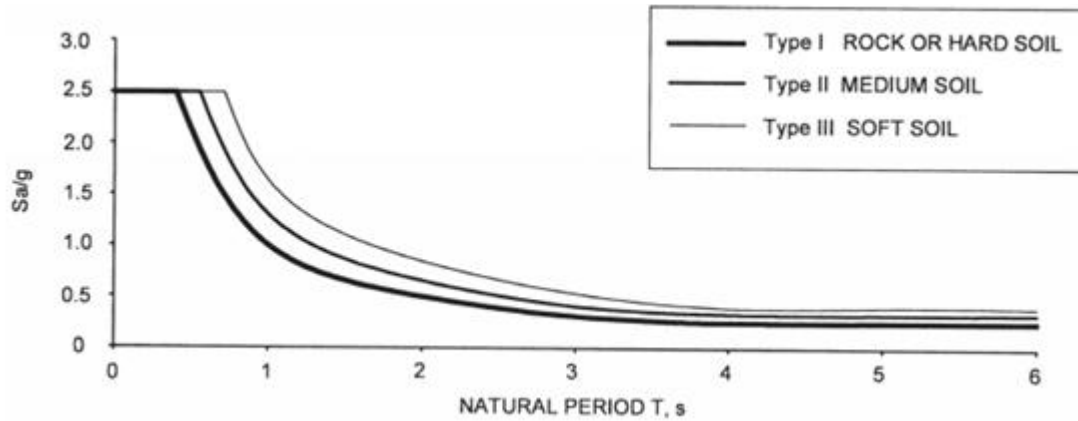


Figure 3. Response Spectra for Equivalent Static method (IS1983:2016)

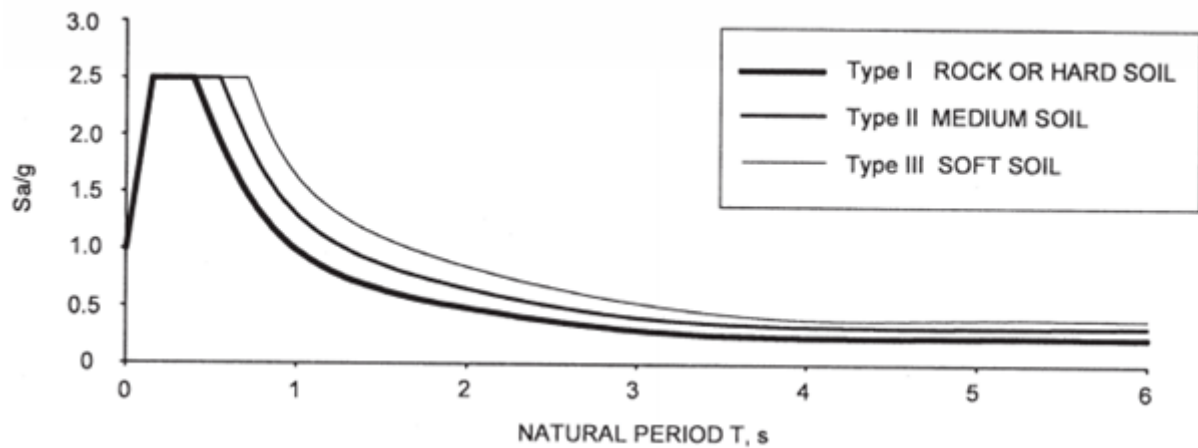


Figure 4. Spectra for Response Spectra Method (IS1893:2016)

The designed horizontal seismic coefficient, A_h for a structure shall be determined by the equation,

$$Eq. 1. \quad A_h = \frac{\left(\frac{z}{L}\right) \left(\frac{S_a}{g}\right)}{\left(\frac{m}{T}\right)}$$

The designed seismic acceleration spectral values, A_v or vertical motion shall be determined by the equation:

$$Eq. 2. \quad A_v = \frac{\left(\frac{Z}{3}\right)\left(\frac{Z}{2}\right)(2.5)}{\left(\frac{m}{I}\right)}$$

Where:

- Z=seismic zone factor, 0.36 for whole of Nepal.
- I=Importance factor, 1.5 for all important hospital buildings and facilities, and 1 for other facilities.
- m=response reduction factor (the IS1893 uses the symbol “R” to represent response reduction factor for new design. However, to differentiate between new design and existing buildings, m, has been included here)
- $\frac{S_a}{g}$ =design acceleration coefficient for different soil types, normalized with peak ground acceleration, corresponding to natural period T of structure as per IS1893 standards.

Determination of lateral forces: The determination of horizontal seismic forces for elastic response of the structure for maximum response of the earthquake should be calculated with unreduced response spectrum using Eq.1 and Eq.2. The value of I and m in such case will be considered 1 and 1 to see the demand on actual response of earthquake in structures. The term m-factor, component demand modification factor will be used to account for expected ductility associated with action.

5. ASSESSMENT OF EXISTING CONDITION

5.1 Overview

Data on the as-built condition of each structure, its components, the site, and adjacent buildings shall be collected in sufficient detail. This information will be used to identify the structural components that form the Lateral Force Resisting System (LFRS) and to identify seismic deficiencies (such as discontinuities in the load path, weak members and connections, building irregularities, and inadequate strength and deformation capacities). It is critical to document thoroughly the building seismic vulnerabilities that are determined from a condition assessment before proceeding to analytical investigations, which will be followed by seismic retrofit.

5.2 Condition Assessment

An as-built condition evaluation should use the following resources:

- Construction documents including: a) plans and specifications, b) engineering analyses and reports, c) log of soil borings and foundation investigations, d) maintenance records covering the life of the building, and e) product literature and test data for components that were used in construction. Data shall be obtained from design drawings that have sufficient information to analyze component demands and calculate component capacities. Design drawings need not be complete, but they shall communicate the configuration of the gravity and LFRS and typical connections with sufficient detail to carry out linear analysis procedures. All the efforts shall be made to collect these documents.
- Interaction: Generally, it is difficult to find documentation listed above for most of the hospital facilities in Nepal. Therefore, it is necessary to interact with the hospital facilities authorities and other staff who may have involved or overseen the construction and repair of the facility. If possible, efforts shall be made to track the design engineer(s), contractors, and supervisors and interview them. A meeting and interaction with the person responsible for maintenance of the facility could reveal many hidden facts about the building, its maintenance history.
- Field observation: Significant time and efforts shall be made for an on-site investigation of the buildings. The assessment shall not solely rely on secondary information and shall involve data collection and confirmation of available information with the active participation of the authority and owners. During the on-site investigation reports and photographs of any exposed conditions and configuration including geotechnical conditions shall be collected.
- Previously collected data: data that is available from previous seismic evaluation of the building
- Information on adjacent buildings, and on all other key issues that are addressed in the Standard, should be obtained through field surveys and review of as-built information.

At least one site visit shall be made to observe the exposed conditions of the building configuration, building components, the site, foundations, and adjacent structures. This site visit should also verify that as-built information that was obtained from other sources accurately represents the existing conditions.

5.3 Assessment Approach

The assessment of a building structure requires an understanding of the likely behavior of the building components and how these are likely to interact with each other. Same applies with the non-building structures. Similarly, non-structural and secondary components also require understanding of their seismic behavior and how these will interact with the principal and secondary building structure.

The nature of the construction of building means that each building is unique in terms of construction, quality of the original workmanship and current condition. .Therefore, it is important that the assessor have an appreciation of how the building was constructed, its current condition, the observed behavior of

similar buildings in previous earthquakes and a holistic view of the factors likely to affect its seismic performance. These issues should be well investigated prior to progressing through the assessment processes outlined in this section.

It is a general recommendation of these guidelines that the capacity of a building should be considered independently from the demands (imposed inertial loads and displacements) placed on it, bringing both together only in the final step of the assessment process.

Past observations in earthquakes indicate that some components of buildings are particularly vulnerable to earthquake shaking and a hierarchy in vulnerability can be identified that can be useful in guiding the assessment process. For example, Figure 5 shows a capacity “chain” for a typical URM building, with component vulnerability decreasing from left to right on the chain. The capacity of the building will be limited by the capacity of the weakest link in the chain, and the ability of each component to fully develop its capacity will typically be dependent on the performance of components to the left of it on the chain. This suggests that the assessment of component capacities should also proceed from left to right in Figure 5. A similar chain could be developed for RC frame buildings.

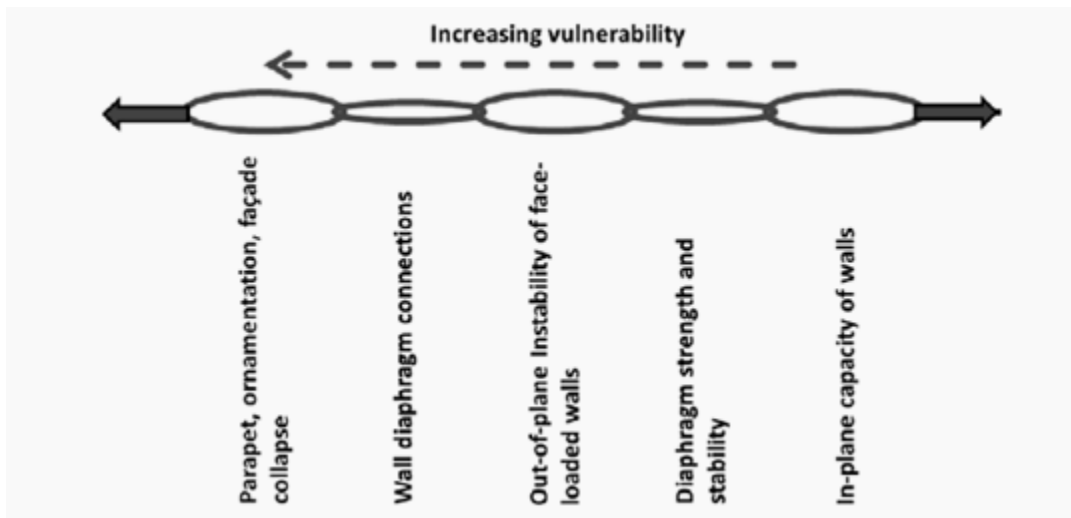


Figure 5. The capacity “chain” and hierarchy of URM building component vulnerability (MBIE, 2017)

While the critical structural weakness in a structural system will often be readily apparent (e.g. lack of any positive ties from brick walls to floors/roof in URM buildings), it will generally be necessary to evaluate the capacity of each link in the chain to fully inform on the components that require retrofit and the likely cost of this. While developing a retrofitting strategy to mitigate the risk, the same chain could be followed from the left to right based on the risk posed.

5.4 Knowledge Factor, κ

The knowledge factor (κ -factor) is used to express the confidence with which the properties of the building components are known when calculating component capacities. In this guideline, a knowledge factor of 0.75 is prescribed. A higher factor can be obtained if the material testing procedure outlined in Standard is conducted.

5.5 Data collection procedure

To obtain a higher knowledge factor than the default value listed in the main body of the guideline, comprehensive data collection including material testing is required. Material testing can be performed during the retrofit construction phase.

5.5.1 Requirements for Comprehensive Data Collection

Comprehensive collection of as-built information should consist of the following:

- Information on adjacent buildings and on all other key issues that are addressed in the Standard should be obtained through field surveys and review of as-built information
- Site and foundation information should be collected
- Information shall be obtained from construction documents, including design drawings, specifications, materials testing records, and quality assurance reports that cover the original construction and subsequent modifications to the structure. When construction documents are available, information shall be verified by a visual condition assessment in accordance with the requirements of this document for various types of construction.
- In absence of the above, default material properties could be used for assessment of the buildings and non-building structures, which should be verified by testing.
- The coefficient of variation in material test results shall be less than 15% ; if not, either use additional testing to lower the coefficient or use the minimum value from the tests. However, while using minimum values, the effect of this should be carefully evaluated through sensitivity analysis.

5.5.2 Knowledge Factor, κ

The knowledge factor (κ -factor) is used to express the confidence with which the properties of the building components are known when calculating component capacities; see Table 2.

Data	Case		
	Yes	Yes	No
Material Testing	Yes	Yes	No
Drawings	No	Yes	No
Material properties	Tests	Documents and tests	Default
Knowledge factor (κ)	0.9	1.0	0.75

Table 2. Knowledge factor[†]

5.6 Materials Sampling

Testing generally is not required on components other than those of the LFRS. If the existing LFRS were being replaced in the retrofit process, materials testing would be required only to quantify the properties of the existing materials at new connection interfaces.

The mechanical properties of concrete components and reinforcement should be determined from available drawings, specifications and other documents in accordance with Section 5.5.1. If such data is available, only limited in situ testing may be required. When existing as-built data is insufficient to determine material properties, such information should be supplemented by materials testing and assessments of existing conditions in compliance with the relevant sections and provisions of the Standard. Mechanical properties for both concrete and reinforcing steel can be established from combined core and reinforcement samples that are taken at similar locations.

The quality of construction and the condition of the materials can significantly influence the existing material properties. In the absence of deleterious conditions or corrosive material, concrete gains compressive strength as it ages, and the existing compressive strength could exceed the specified design values (28-day compressive strength). Therefore, it is likely that for sound concrete, the compressive strength that is determined from samples will exceed the nominal values. Reinforcement continuity

^{††} FEMA 356 was used as reference to derive these values

between existing connecting elements (for example, beams and columns, and diaphragms and shear walls) must also be confirmed.

If additional destructive and nondestructive tests are required to assess the concrete condition, quality and compressive strength, Schmidt Hammer[‡] testing can be used. In that case, Schmidt Hammer readings should be taken at each concrete core location to obtain additional data. Because the results from core samples are more accurate than the values that are obtained from Schmidt Hammer tests, core sample results shall be used when inconsistency between the two data sets exists.

Mechanical properties for masonry materials and components shall be based on available construction documents and on as-built conditions for a particular structure. If these sources fail to provide adequate information to quantify material properties or to document the condition of the structure, such information shall be supplemented by materials testing and assessments of existing conditions.

5.6.1 Concrete

Nondestructive testing to determine the concrete strength and size and location of reinforcement should precede concrete-core sampling and other intrusive methods. Core sampling shall not compromise the strength of the existing structure; in particular, core locations are to be chosen in a way that avoids or minimizes damage to existing reinforcement. Either concrete cubes or cylinders shall be taken. No coring is permitted in columns that have dimensions equal to or less than 250 mm by 250 mm. Cored holes should be filled with concrete or grout of comparable strength.

Core samples should be taken from components that provide resistance to lateral or vertical loading. Samples shall be distributed uniformly in each story. Additional cores should be taken from damaged or deteriorated components, if such elements exist.

The sampling and the minimum number of cores should be based on the following:

- For each concrete element type, a minimum of three core samples shall be taken and be subjected to compression tests.
- A minimum of six tests to determine concrete strength shall be performed on a building, subject to the limitations of this section.
- If varying concrete classes or grades were employed in the building construction, for each class or grade, a minimum of three samples shall be obtained, and testing on each sample shall be performed.
- Samples shall be taken from components, distributed throughout the building, that are critical to the structural behavior of the building.
- Tests shall be performed on samples from components that are identified as damaged or degraded to quantify their condition.
- Test results from areas of degradation shall be compared with the strength values that are specified in the construction documents. If test values lower than the specified strength in the construction documents are found, further strength testing shall be performed to determine the cause or to identify the degree of damage or degradation.
- The minimum number of tests to determine compressive strength shall conform to the following criteria:

[‡] Schmidt hammer is a device used to measure the properties of concrete. The hammer measures the rebound of a spring loaded mass against the concrete surface and is calibrated to measure the compressive strength (Schmide, 1950)

- For concrete elements for which the specified design strength is known and test results are not available, a minimum of three cores/tests shall be conducted for each floor level, for 30 m³ of concrete, or for 900 m² of surface area, whichever requires the most frequent testing; or
- For concrete elements for which the design strength is unknown and test results are not available, a minimum of six cores/tests shall be conducted for each floor level, for 30 m³ of concrete, or for 900 m² of surface area, whichever requires the most frequent testing. Where the results indicate that different classes of concrete were employed, the degree of testing shall be increased to confirm class use.

Concrete cores shall be laboratory-tested to establish the compressive strength (f'_c) of the samples. After the compressive strength is known, the tensile strength and modulus of elasticity can be determined by using available equations.

The mean value of the compressive stresses that is obtained from the testing for each class of concrete shall be used in analysis and evaluation. When sample cores have a coefficient of variation greater than 15%, additional testing should be performed until the coefficient of variation is less than 15%.

5.6.2 Reinforcing Steel

During the on-site surveys, reinforcing-steel classes should be determined. If the nominal design strength of the reinforcing steel is known, additional testing is not required. If the specified design strength of the reinforcing steel is not known, at least one coupon of reinforcing steel should be removed from the building for each class and size of reinforcement. The removed steel should be replaced with a splice bar. The reinforcement sample should be taken from a beam or a shear wall on a basement floor, or from a secondary beam on another floor. The length of the sample should be at least 800 mm. The reinforcement shall be checked for evidence of degradation and corrosion, and any anomalies (for example, a percentage of sectional loss of reinforcement from corrosion) should be documented.

The reinforcement coupons are to be tested to determine their yield and ultimate strengths and elongation. Additional tests are to be conducted to determine the carbon equivalent that is present in the reinforcing steel. The testing laboratory is to provide both numerical (digital) and graphical stress-strain data for each specimen. The tabulated data should include reinforcement diameter, yield and tensile strengths, and percentage of elongation—both as measured from the sample and as indicated by the specified minimum values for that grade of reinforcement.

When as-built data is not available, testing shall also be conducted to determine the size and spacing of transverse reinforcement in concrete columns and beams. Data shall be collected near the midspan and near joints. If the concrete cover is removed, it shall be replaced by concrete of similar strength. If test or as-built data indicates inadequate confinement, additional confinement can be provided by using the retrofit measures of the Standard. Alternatively, nonductile, inelastic behavior shall be assumed in analysis and evaluation.

If data on the length of lap splices for longitudinal reinforcement is not available, testing shall be conducted to determine such length and to determine whether the splices are located in the high seismic demand, no-splice zones. All removed concrete shall be replaced by material of similar strength. If test or as-built data indicates insufficient splice length, the retrofit measures of the Standard can be used for mitigation. Alternatively, reduced strength for longitudinal reinforcement shall be used to account for the short splice length.

5.6.3 Masonry[§]

In the absence of default values and for verification of the used default values, the masonry compressive strength, f_{me} , shall be measured by using one of the following three methods: (1) Test prisms shall be extracted from an existing wall and be tested; (2) prisms shall be fabricated from actual extracted masonry units, and a surrogate mortar shall be designed on the basis of a chemical analysis of actual mortar samples; or (3) for solid unreinforced masonry (URM), the strength of the masonry can be estimated by using a flat-jack test. For each of the three methods that are enumerated in this section, the compressive strength shall be based on the net mortared area.

In the absence of default values and for verification of the used default values, the values of the elastic modulus for masonry in compression, E_{me} , shall be measured by using one of the following two methods: (1) Test prisms shall be extracted from an existing wall and be tested in compression, and stresses and deformations shall be measured to determine modulus values; or (2) for solid URM, the modulus can be measured by using a flat-jack test.

The flexural tensile strength, f_{te} , for out-of-plane bending shall be measured by using one of the following three methods: (1) Test samples shall be extracted from an existing wall and be subjected to minor-axis bending by using the bond wrench method; (2) test samples shall be tested in situ by using the bond wrench method; or (3) sample wall panels shall be extracted and subjected to minor-axis bending. Unless testing is performed to define the tensile strength for in-plane bending, flexural tensile strength for URM walls that are subjected to in-plane lateral forces shall be assumed equal to that for out-of-plane bending.

The masonry shear strength, v_{me} , shall be measured by using an approved in-place shear test. The shear strength shall be determined in accordance with Eq. 3.

$$Eq. 3. \quad v_{me} = \frac{1}{2} \left(0.75v_{te} + \frac{P_{CE}}{A_n} \right)$$

Where:

- P_{CE} = Gravity compressive force that is applied to a wall or a pier component considering the load combinations given in Section 6.5.
- A_n = Area of net mortared/grouted section of a wall or pier.
- v_{te} = Average bed-joint shear strength, v_{t0} , but not greater than 700 kPa:

$$Eq. 4. \quad v_t = \frac{V_{test}}{A_b} - P_G$$

Where:

- V_{test} = Test load at first movement of a masonry unit.
- A_b = Sum of net mortared area of bed joints above and below the test unit.
- P_G = Stress due to gravity loads at the test location.

The shear modulus of masonry (unreinforced or reinforced), G_{me} , shall be taken as 0.4 times the elastic modulus in compression.

Materials testing is not required if material properties are available from original construction documents that include materials test records or materials test reports.

The minimum number of tests to determine masonry material properties for the usual data collection shall be based on the following criteria:

[§] Applies to both brick and stone masonry and for cases that either no mortar is used or when either mud or cement mortar are used.

- For masonry that is in good or fair condition, a minimum of three tests shall be performed for each masonry type, and for each three floors of construction or 300 m² of wall surface, if original construction records are available that specify material properties. If original construction records are not available, six tests shall be performed. At least two tests shall be performed for each wall or line of wall elements that provides a common resistance to lateral forces. A minimum of eight tests shall be performed for each building.
- For masonry that is in poor condition, additional tests shall be performed to estimate material strengths in regions where conditions differ.

Samples for tests shall be taken at locations that represent the material conditions throughout the entire building, taking into account variations in:

- Workmanship at different story levels
- Weathering of the exterior surfaces
- The condition of the interior surfaces due to deterioration caused by leaks and water condensation and/or the deleterious effects of other substances that the building contains.

5.7 Geotechnical Investigation

5.7.1 Soil Classifications

Unless a site investigation is conducted, Soil Class III (soft soil) in accordance with IS1893 shall be assumed for Nepal.

In addition to the Soil Class, the site should be investigated for other geotechnical hazards, such as liquefaction, land instability, rock fall and landslide. Considering the high cost of these investigations, need of these investigations should be judiciously evaluated based on potential and magnitude of hazard, building importance and size.

5.7.2 Foundation Investigation

Typical public buildings in Nepal use shallow isolated or continuous spread footings or use mat foundations. The condition of the foundation can be a determining factor as to whether a building can or should be retrofitted. Foundation repair or retrofits are typically expensive. Limited tilting and cracking in existing buildings are acceptable if settlement has ceased. The existing condition of the foundation should be determined during the planned retrofit investigation phase. The level of this investigation depends on several factors, including the building size, age, occupancy and foundation condition.

The existing foundation data can be determined from the original design sheets that specify the foundation capacity, and from previous geotechnical reports for the site or for other sites in the immediate vicinity. In particular, it is important to establish the type and size of the foundation. Such data is used in the retrofit phase. For example, if a shear wall retrofit were selected, an additional foundation would be required for the base of the wall if no foundation is present or if the existing foundation has inadequate capacity. Available data should be supplemented by field investigations to help establish in situ conditions.

During the implementation phase, concurrent with materials testing, the responsible agency should oversee the opening of inspection pits, both inside and outside the building. These pits are for assessing the existing substructure conditions, including the type of foundation, its depth, its bottom elevation, and whether tie (grade) beams are present. In addition, the presence of ground or surface water that may affect the foundation integrity should be investigated. At least two pits should be excavated for each building. The location of the foundation pits should be marked on the building site plan.

Data collection should be based on an investigation of construction records to map the soil conditions. This data collection could include a visual survey of the foundation excavations, trenching or drilling, a review of historical soils reports, calculations of loads imparted on the foundation, measurements of groundwater level and pore-water pressure, stress measurements in existing tension ribbons, vibration measurements, and materials sampling.

The following structural information shall be obtained for the foundation of a building that is a candidate for seismic retrofit:

- Foundation type and configuration
- Depth of embedment of shallow foundations
- Material composition and construction details

With this information, the bearing capacity of the foundation can be estimated.

Adjacent building development or grading activities can impose loads on or reduce the lateral support of the building under investigation. Field evaluation and existing drawings should be used to clearly assess whether the adjacent structures influence the subject building. This walk-through can also be used to search for evidence of poor foundation performance, such as settlement of building floor slabs and foundations, or differential movement that is evident at adjacent exterior sidewalks.

6. ANALYSIS PROCEDURE AND ACCEPTANCE CRITERIA

6.1 Introduction

The analysis procedure and acceptance criteria stated in this chapter are applicable to both assessment of existing structures and retrofitted structures and components. The specific parameters for the acceptance criteria are presented in the following chapters. These values differ for the existing and retrofitting structures because it is expected that the existing detailing in the buildings do not have adequate ductility, whereas, if the retrofit follows the procedures and ductile detailing specified in this document, a ductile performance would be expected.

6.2 General Requirements

6.2.1 Overview

An analysis of the building shall be performed by using the Linear Static Procedure (LSP) or Response Spectrum Procedure (RSP). If the building contains out-of-plane offsets in the vertical elements of the Lateral Force Resisting System (LFRS), the model shall explicitly account for such offsets in determining the diaphragm demands.

Multidirectional seismic effects shall be considered to act concurrently as required by the relevant Standards. Components of the building shall be designed for combinations of forces, and deformations shall be determined from separate analysis in the x- and y- directions. Components shall be classified as listed in Table 3.

Elements and component	Resist	Check for
Lateral force resisting system (LFRS)	Gravity and seismic	Forces and deformation from seismic and gravity
Gravity force resisting system (GFRS)	Gravity only	Deformation compatibility and gravity loading

Table 3. Classification of structural components

6.2.2 Diaphragms

Diaphragms are defined as horizontal elements that transfer earthquake-induced inertial forces to vertical elements of the LFRS through the collective action of diaphragm components. Diaphragms shall be provided at each level of the structure. The analytical model of the building shall account for the behavior of the diaphragms. Diaphragms are classified as flexible, semi-rigid, or rigid. Diaphragms shall be classified as flexible when the maximum horizontal deformation of the diaphragm along its length is more than twice the average story drift of the vertical elements of the LFRS of the story that is immediately below the diaphragm. Wood and sheet metal floors are generally considered flexible. Diaphragms shall be classified as rigid when the maximum lateral deformation of the diaphragm is less than half the average story drift of the vertical elements of the LFRS of the associated story. Concrete floors are generally considered rigid. Diaphragms that are neither flexible nor rigid shall be classified as semi-rigid. A rod-braced diaphragm is a possible candidate for a semi-rigid diaphragm. For classifying diaphragms, story drift and diaphragm deformations shall be calculated by using the pseudo-lateral force in this document.

6.2.3 Foundation Modeling

The foundation system shall be modeled considering the degree of fixity that is provided at the base of the structure. Depending on the detailing of foundation, either a rigid or a pinned base is to be used.

6.2.4 Multidirectional Seismic Effects

Multidirectional seismic effects shall be considered to act concurrently for buildings that have plan irregularities and rigid or semi rigid diaphragms. All other buildings are to be designed for seismic motions that act non-concurrently in the direction of each principal axis of the building.

6.2.5 Concurrent Seismic Effects

When concurrent multidirectional seismic effects must be considered, horizontally oriented, orthogonal x- and y-axes shall be established. Components of the building shall be designed for combinations of forces, and deformations shall be determined from separate analysis for ground motions in the x- and y-directions.

6.2.6 Component Gravity Loads for Load Combinations

The following actions due to gravity loads, Q_G , shall be considered for combination with actions caused by seismic loads.

When the effects or actions of gravity and seismic loads are additive, the action due to design gravity loads, Q_G , shall be determined in accordance with Eq. 5:

$$\text{Eq. 5.} \quad Q_G = 1.1 (Q_D + Q_L + Q_S)$$

When the effects or actions of gravity and seismic loads are counteracting, the action due to design gravity loads, Q_G , shall be obtained in accordance with Eq. 6:

$$\text{Eq. 6.} \quad Q_G = 0.9Q_D$$

Where:

- Q_D = action due to design dead load; and
- Q_L = action due to design live load, equal to 25% of the unreduced design live load, but not less than the actual live load.
- Q_S = action due to snow load (when applicable)

6.3 Linear Static Procedure (LSP)

The design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements shall be determined in accordance with this section. Buildings shall be modeled with linearly elastic stiffness. The pseudo-lateral force shall be used to calculate internal forces and system displacements due to the design earthquake. Results of the LSP or RSP shall be checked by using the acceptance criteria.

6.3.1 Limitations

Equivalent static analysis is not allowed in seismic zone V as per IS 1893:2016(Part I), major change in this current code. Hence, response spectrum analysis is recommended for all structures.

6.3.2 Period Determination

The building fundamental period, T , in the direction under consideration shall be determined from:

$$\text{Eq. 7.} \quad T = \frac{0.09}{\sqrt{d}} h$$

Where:

- d is the base dimension of the building
- h is the height of building measuring from the base of the building

This equation applies to concrete moment frame with infills, concrete walled buildings, and brick/stone masonry bearing wall buildings.

Alternatively, the period of the building may be computed from eigenvalue (modal) analysis.

6.3.3 Design base shear

The design base shear in a given horizontal direction of a building shall be determined by using Eq. 8

$$\text{Eq. 8.} \quad V = C_1 A_h W$$

Where:

- V = Pseudo-lateral force
- C_1 = modification factor to relate the expected maximum inelastic displacements to displacements that are calculated for linear elastic response.
 - $C_1 = 2.0$ for $T \leq 0.1$
 - $C_1 = 1.0$ for $T \geq 0.6$
 - Use linear interpolation for the intermediate values of T

Where:

- T = fundamental period (in seconds) of the building in the direction under consideration, and
- A_h = Horizontal seismic coefficient at the fundamental period of the building in the direction under consideration, see Chapter 4.
- W = Effective seismic weight of the building, including the total dead load and applicable portions of live loading.

A factor of 0.75 may be included in the Equation 8. Considering the limited life left of the existing hospital facilities, it is recommended to assess, and design retrofit of the existing hospital facilities for 75% of the seismic force that would be required for a similar new building.

6.3.4 Vertical Distribution of Force

For Hospital buildings, the total force shall be distributed over the height of the structure according to Eq. 9.

$$\text{Eq. 9.} \quad F_x = \frac{w_x h_x}{\sum_{i=1}^n w_i h_i} V$$

Where:

- F_i , w_i and h_i are the seismic lateral force, seismic weight and elevation above base of floor i , respectively

At each level designated as x , the force F_x shall be applied over the area of the building in accordance with the mass distribution at that level. Structural displacements and design seismic forces shall be calculated as the effect of the force F_x applied at the appropriate levels above the base.

6.3.5 Horizontal Distribution of Shear

The design story shear, V_x , in any story is the sum of the forces F_x above that story. V_x shall be distributed to the various vertical elements of the LFRS in proportion to their rigidities, considering the rigidity of the diaphragm.

6.4 Response spectrum procedure (RSP)

When the LSP is not permitted due to limitations, RSP shall be performed to obtain the base shear and its distribution to different level along height and to various lateral load-resisting element. Sufficient numbers of modes shall be included in the RSP to ensure that at least 90% of seismic mass is captured in analysis in both principal directions and in torsion. The response spectrum used in analysis shall be scaled to account for the importance of the hospital; using the I_h factor of Section 6.3.3

6.5 Acceptance Criteria

Components that are analyzed by using linear procedures shall satisfy the requirements of this section. Before component acceptance criteria are selected, components shall be classified as primary or secondary.

Design actions, Q_{UD} , shall be calculated in accordance with Eq. 10.

$$Eq. 10. \quad Q_{UD} = Q_G \pm Q_E$$

Where:

- Q_E = action due to design earthquake loads;
- Q_G = action due to design gravity loads; and
- Q_{UD} = design action due to gravity loads and earthquake loads.

Components shall satisfy:

$$Eq. 11. \quad m\kappa Q_{CE} \geq Q_{UD}$$

Where:

- m = component demand modification factor to account for the expected ductility that is associated with this action at the Life Safety Structural Performance Level;
- Q_{CE} = strength of the component at the deformation level under consideration determined considering all coexisting actions on the component under the design loading condition; and
- κ = knowledge factor.

6.5.1 Story Drifts

A static, elastic analysis of the LFRS shall be conducted by using the unreduced design seismic forces from Section 6.3. The resulting deformations, denoted as δ , shall be determined at all critical locations in the structure. The calculated drift shall include translational and torsional deflections. The calculated story drift shall not exceed 1.5%. The selection of the 1.5% limit is based on the current code provisions. Building codes typically allow for drift ratios of up to 2.0%. However, this limit is based on the assumption of ductile behavior, which might not be present for buildings in Nepal. Accordingly, a lower value is selected in this document.

6.5.2 Deformation Compatibility

All existing GFRS (framing elements and connections that are not required by design to be part of the LFRS) shall be designed and detailed to maintain support of the design dead load plus the live load when subjected to the expected deformations caused by seismic forces. The requirements of this section are considered satisfied if the story drift ratio (δ/h) does not exceed 1.0%. The choice of 1.0% limit is because at this level of drift, it is anticipated that the structural components will remain elastic.

7. ASSESSMENT OF REINFORCED CONCRETE BUILDINGS

7.1 Overview

Reinforced buildings constructed per requirements of modern seismic code are expected to perform well in earthquakes and provide life protection. By contrast, older concrete buildings or buildings without the ductile detailing of reinforcement are one of the most dangerous construction types and have resulted in many collapsed buildings and loss of thousands of lives in the recent earthquakes around the world. In this chapter, the procedure for the assessment of the reinforced concrete hospital buildings is presented.

The masonry infill walls are invariably present in most RC frame buildings in Nepal. As observed in past earthquakes including the 2015 Nepal earthquake, these infill walls bring significant uncertainty to the seismic performance of buildings. A few of the ill effects that these infill walls could impose are configurational deficiencies, short column effect, soft/weak story mechanism, etc.

The current practice in Nepal is to ignore these walls while assessing building structures. This could lead to a dangerously different conclusion than how the building actually performs under seismic shaking. Hence, the effect of these walls shall be included in the seismic assessment of the RC frame building structures with infill walls.

7.2 General Procedure for the Evaluation of Reinforced Concrete Buildings

7.2.1 Scope

This chapter sets forth requirements for the systematic retrofit of concrete components of the LFRS of an existing building. The requirements of this chapter shall apply to 1) existing concrete components of a building system, 2) rehabilitated concrete components of a building system and 3) to new concrete components that are added to an existing building system.

7.2.2 Material Properties

Mechanical properties of concrete materials and components shall be ascertained from available drawings, specifications, other documents for the existing construction, and material properties of similar buildings of the era and from testing.

The following component and connection material properties shall be determined for the as-built structure:

- Concrete compressive strength
- Yield and ultimate strength of reinforcing steel

The following component properties and as-built conditions shall be established:

- Cross-sectional dimensions of individual components and overall configuration of the structure
- Configuration of component connections, size of anchor bolts, thickness of connector material, anchorage and interconnection of embedment, and the presence of bracing or stiffening components
- Modifications to components or the overall configuration of the structure
- Current physical condition of components and connections, and the extent of any deterioration present
- Presence of conditions that influence building performance

A knowledge factor, κ , for computing concrete component capacities shall be selected in accordance with this document, per Section 5.4. In lieu of available design specifications or material testing, conservative default values based on construction vintage may be considered.

7.2.3 Condition Assessment

A condition assessment of the existing building and the site shall be performed as specified in this section.

The condition assessment shall include the following:

- The physical condition of lateral and gravity load resisting components shall be examined, and the presence of any degradation shall be noted
- The presence and configuration of components and their connections, and the continuity of load paths between components, elements and systems shall be verified or established
- Other conditions—including neighboring party walls and buildings, the presence of nonstructural components, prior remodeling, and limitations for rehabilitation—that may influence building performance shall be reviewed and documented
- Information that is necessary to select a knowledge factor shall be obtained
- Component orientation, plumbness and physical dimensions shall be confirmed

The results of the condition assessment shall be used to quantify the following items, which are needed to create the mathematical building model:

- Component section properties and dimensions
- Component configuration and the presence of any eccentricities or permanent deformation
- Connection configuration and the presence of any eccentricities
- Presence and effect of alterations to the structural system since the original construction
- Interaction of nonstructural components and their involvement in lateral-load resistance

All deviations between available construction records and as-built conditions that are noted from visual inspection shall be accounted for in the structural analysis.

Unless concrete cracking, reinforcement corrosion or other mechanisms are observed in the condition assessment as causing damage or reduced capacity, the cross-sectional area and other sectional properties shall be taken as those from the design drawings and site measurements or from tests. If some sectional material loss has occurred, the loss shall be quantified by direct measurement, and sectional properties shall be reduced accordingly, using the principles of structural mechanics.

7.2.4 Modeling and Design

Modeling and analysis of structural components of existing buildings shall comply with the requirements of this document. Evaluation of the demands and capacities of reinforced concrete components shall also consider locations along the length of the components where lateral and gravity loads produce maximum effects; where changes in the cross section or reinforcement result in reduced strength; and where abrupt changes in the cross section or reinforcement, including splices, might produce stress concentrations, resulting in premature failure.

7.2.4.1 Stiffness

Cracked component stiffness shall be calculated considering flexure and shear behavior; see Table 4.

Component	Flexural rigidity	Shear rigidity
Beams	0.35	0.4
Columns	0.50	0.4
Walls	0.50	0.4
Flat slabs	–	0.4

Table 4. Effective stiffness values accounting for cracked properties**

7.2.4.2 Strength

The strengths, Q_{CE} , are calculated by using accepted principles of mechanics. Strength and deformation capacities shall be determined considering the available development of longitudinal reinforcement. For concrete columns that are under combined axial load and biaxial bending, the combined strength shall be evaluated considering biaxial bending.

For beams and columns, shear and torsional strength shall be calculated, based on the maximum moment developed by the members.

- When the longitudinal spacing of transverse reinforcement exceeds half the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed as not more than 50% effective in resisting shear or torsion.
- When the longitudinal spacing of transverse reinforcement exceeds the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed as ineffective in resisting shear or torsion.
- For beams and columns, lap-spliced transverse reinforcement shall be assumed as ineffective.

When longitudinal reinforcement has an embedment or development length that is insufficient to develop reinforcement strength, flexural strength shall be calculated based on limiting the stress capacity of the embedded bar.

- Deformed straight bars, lap-spliced bars shall have a development length as specified in the relevant national concrete standard. This development length is reduced when hooked bars are used. When existing deformed bars do not meet the development requirements mentioned in the preceding paragraph, the capacity of the existing reinforcement shall be calculated by using Eq. 12:

$$Eq. 12. \quad f_s = \left(\frac{l_b}{l_d} \right) f_y$$

Where:

- f_s = maximum stress that can be developed in the bar for the straight development, hooked development, or lap splice length, l_b , provided;
- f_y = yield strength of reinforcement; and
- l_d = required development length
- l_b = provided length

This capacity, however, shall not exceed the yield strength.

- For plain straight bars, hooked bars, and lap-spliced bars, the development and splice lengths shall be taken as suggested by relevant national Standards.

**** FEMA 356 was used as reference to derive these values

- Post-installed dowel bars added in seismic rehabilitation shall be assumed to develop yield stress when all the following conditions are satisfied:
 - Drilled holes for dowel bars are cleaned with a stiff brush that extends the length of the hole.
 - The embedment length, l_e , is not less than $10d_b$, where d_b is the bar diameter.
 - The minimum spacing of dowel bars is not less than $4l_e$, and the minimum edge distance is not less than $2l_e$. Design values for dowel bars that do not satisfy these conditions shall be verified by test data. Field samples shall be obtained to ensure that design strengths are developed.

Square reinforcing bars in a building shall be classified as either twisted or plain. The developed strength of twisted square bars shall be as specified for deformed bars, using an effective diameter that is calculated based on the gross area of the square bar. Plain square bars shall be considered as plain bars, and the developed strength shall be as specified for plain bars.

7.2.4.3 Deformation and ductility

The demand/yield ratio on existing concrete members shall be limited as stated in this standard. Retrofit measures shall be evaluated in accordance with the requirements of this document to ensure that the completed retrofit achieves the selected upgrade goal. The effects of retrofit on stiffness, strength and deformability shall be taken into account in an analytical model of the structure. The compatibility of new and existing components shall be checked, unless the story drift ratio is limited to 1.0% or less.

7.2.5 Reinforced Concrete Moment Frames with brick or stone masonry infills

7.2.5.1 Overview

Concrete frames with infills are complete gravity-load-carrying concrete frames that are infilled with masonry or stone, and constructed in such a way that the infill and the concrete frame interact when they are subjected to vertical and lateral loads. The infill might have mud or cement mortar, or might have no mortar. The provisions of this section shall apply to concrete infills that interact with concrete frames, where the infills were constructed to fill the space within the bay of a complete gravity frame without special provisions for continuity from story to story.

Concrete moment frames shall be defined as elements that comprise horizontal framing components (beams), vertical framing components (columns), and joints that connect the beams and columns. Beams and columns shall be of monolithic construction that provides for moment transfer between beams and columns.

7.2.5.2 General Considerations

The analytical model for a concrete frame with infills shall represent the strength, stiffness and deformation capacity of beams, slabs, columns, beam-column joints, infills, and all connections and components of the elements. Potential failure in flexure, shear, anchorage, reinforcement development or crushing at any section shall be considered. Interaction with other nonstructural elements and components shall be included.

The analytical model shall be established considering the relative stiffness and strength of the frame and the infill, as well as the level of deformation and associated damage. The infill can be modeled as a compression struts with effect of openings included where they occur.

Frame components shall be evaluated for actions that are imposed on them through interaction of the frame with the infill. For frames with partial-height infills, the evaluation shall include the reduced effective length of the columns above the infilled portion of the bay. The resulting captive column must be addressed.

Where infills create a discontinuous wall, the effects of the discontinuity on overall building performance shall be evaluated.

The analytical model for a beam-column frame element shall represent the strength, stiffness, and deformation capacity of beams, columns, beam-column joints, and other components of the frame, including connections with other elements. Potential failure in flexure, shear, and reinforcement development at any section along the component length shall be considered. Interaction with other elements, including nonstructural components, shall be included. Analytical models representing a beam-column frame that uses line elements with properties concentrated at component centerlines are permitted. If beam and column centerlines do not intersect, the effects of the eccentricity between the centerlines of the framing shall be taken into account.

7.2.5.3 Stiffness

For frames that have infill in some bays and no infill in other bays, the restraint of the infill shall be represented, and the non-infilled bays shall be modeled as frames. Beams shall be modeled considering flexural and shear stiffness, including the effect of the slab acting as a flange in monolithic construction. Columns shall be modeled considering flexural, shear, and axial stiffness. Effective stiffness shall be used in accordance with Table 4. Joint stiffness can be modeled implicitly by using centerline dimension for beams and columns.

7.2.5.4 Strength and Capacity

The strengths of reinforced concrete components shall be calculated according to the general requirements of the relevant National Code, ACI 318 (2014) or equivalent. The maximum component strength shall be determined considering potential failure in flexure, axial load, shear, torsion, development, and other actions at all points along the length of the component, under the actions of design gravity and earthquake load combinations.

Strengths of infills shall be calculated according to the requirements of Chapter 8. Strength calculations shall consider:

Limitations imposed by beams, columns and joints in non-infilled portions of frames

Tensile and compressive capacity of columns acting as boundary elements of infilled frames

Local forces applied from the infill to the frame

Strength of the infill

Connections with adjacent elements

7.2.5.5 Acceptance Criteria

Design actions shall be compared with design strengths; m -factors shall be selected from Table 5 through Table 7. When the average demand-capacity ratio (DCR) for columns at one level exceeds the average value for beams at the same level, and exceeds the greater of 1.0 or $m/2$ for all columns, the level shall be defined as a weak story. The structure shall be retrofitted to remove weak stories.

Case	m-factors
Beams controlled by flexure	3
Beams controlled by any of the following: a) shear, b) inadequate development or splicing of reinforcement along the span, and c) inadequate embedment (development) into beam-column joint	1

Table 5. Numerical acceptance criteria for reinforced concrete beams^{††}

Case	m-factors
Columns controlled by flexure	2
Columns controlled any of the following: a) shear, b) inadequate development or splicing of reinforcement, and c) axial load exceeding $0.70A_g f'_c$	1

Table 6. Numerical acceptance criteria for reinforced concrete columns^{††}

Case	m-factors
All joints	1

Table 7. Numerical acceptance criteria for reinforced concrete joints^{§§}

7.2.6 Reinforced Concrete Shear Walls

7.2.6.1 Overview

Shear walls can be considered as solid walls if they have openings that do not significantly influence the strength or inelastic behavior of the walls. Monolithic reinforced concrete shear walls shall consist of vertical cast-in-place elements. These walls shall have relatively continuous cross sections and reinforcement, and shall provide both vertical- and lateral-force resistance.

Shear walls shall be permitted to resist seismic forces only if all of the following requirements are met: a) axial loads less than $0.35A_g f'_c$, b) spacing of horizontal and vertical reinforcement not exceeding 450 mm, and c) horizontal, and vertical reinforcement ratios not less than 0.0025.

7.2.6.2 General Considerations

The analytical model for a reinforced concrete shear wall element shall represent the stiffness, strength and deformation capacity of the shear wall. Potential failure in flexure, shear and reinforcement development at any point in the shear wall shall be considered. Interaction with other structural and nonstructural components shall be included. The diaphragm action of concrete slabs that interconnect shear walls and frame columns shall be represented in the model.

7.2.6.3 Stiffness

The effective stiffness of all the elements shall be based on the effective stiffness values in Table 4. In using linear analytical procedures, shear walls and associated components shall be modeled considering axial, flexural, and shear stiffness.

^{††††} FEMA 356 was used as reference to derive these values

^{††††} FEMA 356 was used as reference to derive these values

^{§§§§} FEMA 356 was used as reference to derive these values

7.2.6.4 Strength

Component strengths shall be computed according to the general requirements of the National Code, ACI 318 (2014) or equivalent. Strength shall be determined considering the potential for failure in flexure, shear or development under combined gravity and lateral loads.

7.2.6.5 Acceptance Criteria

When determining the appropriate value for the design actions, it is necessary to take into account gravity loads and the maximum forces that can be transmitted between adjacent components. Design actions shall be compared with design strengths; m -factors shall be selected from Table 8.

Case	m -factors
Cast-in-place walls	2

Table 8. Numerical acceptance criteria for walls controlled by flexure or shear ***

7.2.7 Cast-in-Place Concrete Diaphragms

7.2.7.1 Components of cast-in-place Concrete Diaphragms

Cast-in-place concrete diaphragms transmit inertial forces within a structure to the vertical elements of the LFRS. Concrete diaphragms shall consist of slabs, struts, collectors, and chords. Alternatively, diaphragm action may be provided by a structural truss in the horizontal plane. Diaphragms that consist of structural concrete topping on metal deck shall also comply with the requirements of the Standard.

7.2.7.1.1 Slabs

Slabs shall consist of cast-in-place concrete systems that, in addition to supporting gravity loads, transmit inertial loads that have developed within the structure from one vertical element of the LFRS to another. They shall also provide out-of-plane bracing to other portions of the building.

7.2.7.1.2 Struts and Collectors

Collectors are components that transmit the inertial forces within the diaphragm to elements of the LFRS. Struts are components of a structural diaphragm that are used to provide continuity around an opening in the diaphragm. Struts and collectors shall be monolithic with the slab, occurring within either the slab thickness or a thickened slab region.

7.2.7.1.3 Diaphragm Chords

Diaphragm chords are components along diaphragm edges with increased longitudinal and transverse reinforcement, acting primarily to resist tension and compression forces that are generated by bending in the diaphragm. Exterior concrete walls are permitted to serve as chords, if there is adequate strength to transfer shear between the slab and the wall.

7.2.7.2 General Considerations

The analytical model for a diaphragm shall represent the strength, stiffness and deformation capacity of each component and the diaphragm as a whole. Potential failure in flexure, shear, buckling and reinforcement development shall be considered. Modeling of the diaphragm as a continuous or simple-span horizontal beam that is supported by elements of varying stiffness is permitted.

***** FEMA 356 was used as reference to derive these values

7.2.7.3 Stiffness

Diaphragm stiffness shall be modeled by using a linear elastic model and gross section properties. The modulus of elasticity used shall be that of the concrete as specified in this document. The effects of diaphragm flexibility shall be considered when the length-to-width ratio of the diaphragm exceeds 2.

7.2.7.4 Strength

Component strengths shall be computed according to the general requirements of the relevant national standard or equivalent. The maximum component strength shall be determined considering potential failure in flexure, axial load, shear, torsion, development, and other actions at all points in the component when it is under the actions of design gravity and lateral-load combinations.

7.2.7.5 Acceptance Criteria

Design actions shall be compared with design strengths; *m*-factors shall be selected from Table 9.

Case	m-factors
Slabs	2
Other components and connections	1

Table 9. Numerical acceptance criteria for slab components^{†††}

^{†††††} FEMA 356 was used as reference to derive these values

8. ASSESSMENT OF UNREINFORCED BRICK OR STONE MASONRY BEARING WALL BUILDINGS

8.1 Scope

The requirements of this chapter shall apply to brick or dressed and semi-dressed stone masonry walls. Because of the inherent geological weaknesses in construction of stone masonry walls, the stone masonry walls suffer very distinct (such as delamination of wythes, slumping of wall, mechanism failure, etc.) failure mechanism, which makes them far more vulnerable than brick masonry buildings. These issues shall be carefully understood and incorporated in the assessment process.

8.2 Material Properties

Existing construction documents or testing per Section 5.6.3 may be used to determine the material properties of masonry. In lieu of available design specifications or material testing, conservative default values based on construction vintage may be considered. However, to arrive at any reliable judgement, some on-site testing such as scratching, etc. as discussed in this section is recommended. A knowledge factor, κ , shall be selected per Section 5.4.

This section provides default probable material properties for masonry and other associated materials. These values can be used for assessment of URM buildings in the absence of a comprehensive testing program.

Recommended probable default material properties for clay bricks and lime/cement mortar, correlated against hardness, are given in Table 10 and Table 11. The descriptions in these tables are based on the use of a simple scratch test, but there are a variety of similar, simple on-site tests the engineer can use.

Brick hardness	Brick description	Probable brick compressive strength, f_b (MPa)	Probable brick tensile strength, f_t (MPa)
Soft	Scratches with aluminium pick	14	1.7
Medium	Scratches with 10 cent copper coin	26	3.1
Hard	Does not scratch with above tools	35	4.2

Table 10. Probable strength parameters for clay bricks (Almesfer et al., 2014)

Mortar hardness ⁺⁺⁺	Mortar description	Probable mortar compressive strength, f_j (MPa) ^{§§§}	Probable cohesion, c (MPa)	Probable coefficient of friction, μ_f ^{@****}
Very soft	Raked out by finger pressure	0-1	0.1	0.3
Soft	Scratches easily with fingernails	1-2	0.3	0.3
Medium	Scratches with fingernails	2-5	0.5	0.6
Hard	Scratches using aluminium pick	From testing	0.7	0.8
Very hard	Does not scratch with above tools	To be established from testing		0.8

Table 11. Probable strength parameters for lime/ cement mortar (Almesfer et al., 2014)

For limestones a typical compressive strength of 20MPa could be assumed.

⁺⁺⁺When very hard mortar is present it can be expected that walls subjected to in-plane loads and failing in diagonal shear will form diagonal cracks passing through the bricks rather than a stair-stepped crack pattern through the mortar head and bed joints. Such a failure mode is non-ductile. Very hard mortar typically contains cement

^{§§§}From MBIE, 2017

^{****}Values higher than 0.6 may be considered with care/investigation depending upon the nature/roughness of the brick material and the thickness of the mortar with respect to the brick roughness.

8.2.1 Comprehensive strength of masonry

In absence of comprehensive test results, Eq 13 and 14 could be used for estimating compressive strength of masonry. The formulation is verified for brick masonry walls.

$$\text{Eq. 13. } f_{me} = 0.75 f_b^{0.75} f_j^{0.3} \text{ for } f_j \geq 1 \text{ MPa}$$

$$\text{Eq. 14. } f_{me} = 0.75 f_b^{0.75} \text{ for } f_j < 1 \text{ MPa}$$

8.2.2 Tensile strength of masonry

In the absence of any reliable test data, tensile strength of masonry in both horizontal and vertical direction shall be assumed to be zero.

8.2.3 Diagonal tensile strength of masonry

Where specific material testing is not undertaken to determine probable masonry diagonal tension strength, this may be taken as:

$$\text{Eq. 15. } f'_{dt} = 0.5 c + f_a \mu_f$$

Where:

c = masonry bed-joint cohesion

μ_f = masonry co-efficient of friction

f_a = axial compression stress due to gravity loads calculated at the midheight of the wall/pier (MPa).

8.2.4 Modulus of elasticity of masonry

Unless test information is available the modulus of elasticity of the masonry should be calculated using

$$\text{Eq. 16. } E_m = 550 f'_m$$

Where:

f'_m = compressive strength of masonry

8.3 Condition Assessment

A condition assessment per Section 5.2 shall include the following:

- The physical condition of components shall be examined, and the presence of any degradation shall be noted. The condition of the existing masonry shall be evaluated for unit surface or mortar joint deterioration from weathering caused by frequent moisture saturation.
- The presence and configuration of components and their connections, and the continuity of load paths between components, elements, and systems shall be verified or established.

8.4 Engineering Properties of Masonry Walls

The procedures set forth in this chapter for determining stiffness, strength and deformation of masonry walls shall be applied to building systems that comprise any combination of existing masonry walls, masonry walls enhanced for seismic retrofit, and new walls added to an existing building for seismic retrofit.

Masonry walls shall be capable of resisting forces that are applied parallel to their plane and normal to their plane, as described in this chapter. Existing masonry walls shall include all structural walls of a building system that are in place before seismic rehabilitation. Existing masonry walls shall be assumed to behave in the same manner as new masonry walls, if the masonry is in fair or good condition. New

masonry walls shall include all new wall elements that are added to an existing LFRS. Enhanced masonry walls shall include existing walls that are retrofitted by an approved method.

8.5 Unreinforced Masonry Walls and Piers—In Plane

8.5.1 General Consideration

The engineering properties of unreinforced masonry (URM) walls that are subjected to lateral forces that are applied parallel to the wall plane shall be determined in accordance with this section. Requirements of this section shall apply to cantilevered masonry walls that are fixed against rotation at their base and shall apply to piers between window or door openings.

8.5.2 Stiffness

The stiffness of a URM wall or pier resisting lateral forces that are parallel to its plane shall be considered proportional with the geometrical properties of the uncracked section. Story shears in perforated shear walls shall be distributed to piers in proportion to the relative lateral uncracked stiffness of each pier. Stiffness for existing and enhanced walls shall be determined by using the principles of mechanics accounting for both flexure and shear deformations.

8.5.3 Strength

Masonry walls are either unpenetrated or penetrated. A penetrated wall consists of piers between openings plus a portion below openings (sill masonry) and above openings (spandrel masonry). When subjected to in-plane earthquake shaking, masonry walls and piers may demonstrate diagonal tension cracking, rocking, toe crushing, sliding shear, or a combination of these. Similarly, the spandrels may demonstrate diagonal tension cracking, unit cracking or joint sliding. Figure 6 shows the potential failure mechanisms for unpenetrated and penetrated walls.

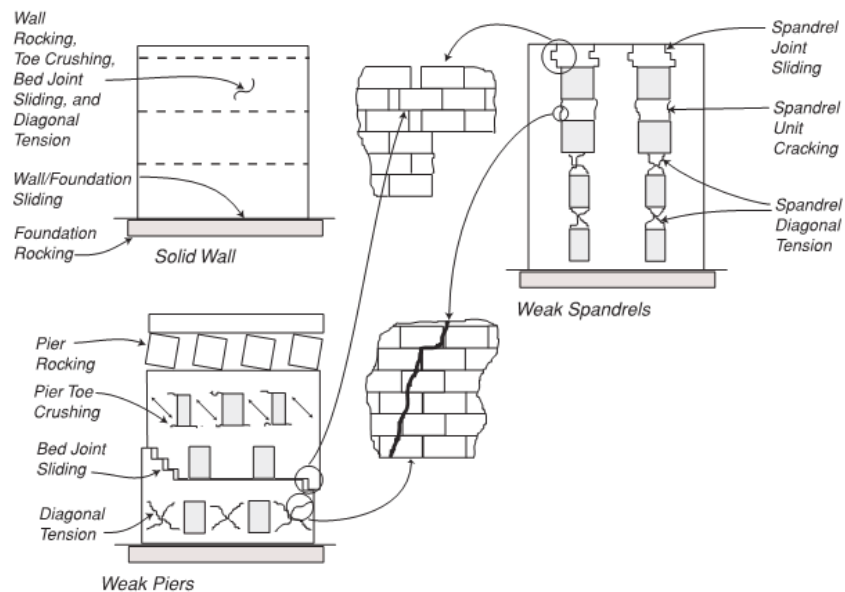


Figure 6. In-plane failure modes of URM wall (FEMA 2000)

The lateral strength, Q_{CE} , of existing URM walls or piers shall be estimated using the Equations 16 to 19:

$$\text{Eq. 17. } Q_{CE} = V_{bjs} = 0.7 (c A_n + \mu_f (P + P_w)) \quad \text{Bed - joint sliding}^{\dagger\dagger\dagger}$$

The factor 0.7 has been introduced to reflect the overall reliability of the sliding mechanism calculation.

$$\text{Eq. 18. } Q_{CE} = V_r = 0.9(\alpha P_E + 0.5P_w) \frac{L}{h_{eff}} \quad \text{Rocking}$$

Bed joint sliding and rocking of the walls and piers are considered stable mode (non-brittle) of failure.

$$\text{Eq. 19. } Q_{CE} = V_{tc} = (\alpha P_E + 0.5P_w) \left(\frac{L}{h_{eff}}\right) \left(1 - \frac{f_a}{0.7f'_m}\right) \quad \text{Toe crushing}$$

$$\text{Eq. 20. } Q_{CE} = V_{dt} = f'_{dt} A_n \beta \sqrt{\left(1 + \frac{f_a}{f'_{dt}}\right)} \quad \text{Diagonal tensile}^{\#\#\#\#}$$

Similarly, toe crushing and diagonal tensile failure are considered unstable modes of failure as they lead to high degradation of the masonry under repeated cycles of loading.

Where:

- A_n = Area of net mortared/grouted section
- h_{eff} = Height to resultant of lateral force
- L = Length of wall or pier
- P_E = Superimposed axial compressive force due to gravity loads
- P_w = Self-weight of wall pier
- f_a = Axial compressive stress caused by gravity loads on pier
- α = Factor equal to 0.5 for a fixed-free cantilevered wall or equal to 1.0 for a fixed-fixed pier
- β = 0.67 for $L/h < 0.67$, L/h when $0.67 \geq L/h \leq 1.0$ and 1 when $L/h > 1.0$.
- f'_{dt} = Diagonal tensile strength of masonry

Refer to Figure 7 for the symbols used in the above formulations:

^{†††}MBIE, 2017

^{\#\#\#\#}ASCE 41-2013

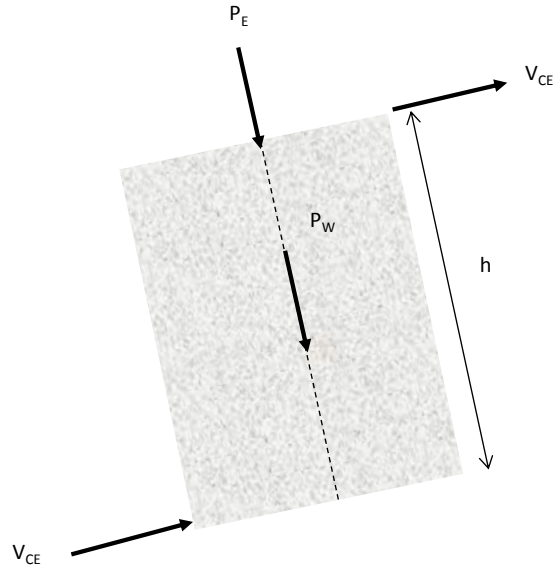


Figure 7. In-plane failure modes of URM wall

Where:

- A_n = Area of net mortared/grouted section
- h = Height to resultant of lateral force
- L = Length of wall or pier
- P_E = Axial compressive force due to gravity loads
- v_{me} = Bed-joint sliding shear strength
- α = Factor equal to 0.5 for a fixed-free cantilevered wall or equal to 1.0 for a fixed-fixed pier.

8.5.4 Mixed Mode Failure Mechanism

When there are mixed behavior modes among the walls/piers in a line of resistance, the engineer must take the mechanism with the lowest m-factor (refer Table 10) to define the m-factor for that line as a whole. Alternatively, the capacity of any piers for which m is less than the value that has been adopted for the line of resistance can be ignored; but only if the consequences of loss of gravity load support from these walls/piers does not cause instability to any of the structure above.

If there are mixed failure modes among the walls and piers in a line of resistance, the displacement compatibility between these piers and walls should be evaluated.

8.5.5 Acceptance Criteria

. The m-factors to use shall be obtained from Table 12.

Case	m-factors	Notes
Bed-joint sliding, stairstep failure modes	2	Failure dominated by strong brick-weak mortar
Rocking	2	Failure dominated by strong brick-weak mortar
Toe crushing	1.0	Failure dominated by weak brick-strong mortar
Diagonal tensile failure	1.0	Failure dominated by weak brick-strong mortar

Table 12. Numerical acceptance criteria for URM walls ^{§§§§}

§§§§§§§§ FEMA 356 was used as reference to derive these values

8.6 Unreinforced Masonry Walls—Out-of-Plane

8.6.1 Walls supported at both top and bottom

8.6.1.1 General

URM walls shall be evaluated for out-of-plane inertial forces as isolated components that span between floor levels and/or that span horizontally between columns or pilasters.

8.6.1.2 Stiffness

The out-of-plane stiffness of walls shall not be included in analytical models of the global structural system in the orthogonal direction.

8.6.1.3 Acceptance Criteria

Stability need not be checked for walls that span vertically with a height-to-thickness (h/t) ratio less than that given in Table 13, if effective wall to diaphragm connections and diaphragm stiffness are present.

Wall type	High seismic zone
All	13

Table 13. Maximum h/t ratios

8.6.2 Cantilever walls

URM walls such as parapets shall be evaluated for out-of-plane inertial forces as free-standing cantilevers.

8.6.2.1 Acceptance Criteria

Stability need not be checked for walls that span vertically with a height-to-thickness (h/t) ratio less than that given in Table 14.

Wall type	High seismic zone
All	1.5

Table 14. Maximum h/t ratios for parapets

***** FEMA 356 was used as reference to derive these values

9. ASSESSMENT OF NONSTRUCTURAL COMPONENTS

9.1 General

Elements of structures and their attachments, permanent nonstructural components and their attachments, and attachments for permanent equipment supported by structure services pipelines shall be designed to resist the total design seismic forces prescribed in this section.

Attachments shall include anchorage and required bracing, however. Friction resulting from gravity loads shall not be considered as providing resistance to seismic forces.

When the structural failure of the lateral-force-resisting systems of non-rigid equipment would cause a life-safety hazard, such systems shall be designed to resist the seismic forces.

When permissible design strengths and other acceptance criteria are not contained in or referenced by this Standard, such criteria shall be obtained from approved national standards or relevant international standards, subject to the approval of a building official.

9.1.1 Design for Total Lateral Force

The total design lateral seismic force, F_p , shall be determined from Eq. 21.

$$\text{Eq. 21.} \quad 0.7(PGA)I_p W_p \leq F_p = \frac{a_p (PGA)I_p}{R_p} (1 + 3 \frac{z_x}{h_x}) W_p \leq 4.0(PGA)I_p W_p$$

Where:

- W_p is the seismic weight of the component
- I_p is the importance factor for the component
- PGA from Chapter 4
- h_x = element or component attachment elevation with respect to grade. The value of h_x shall not be taken as less than 0.0;
- h_r = structure roof elevation with respect to grade;
- a_p = in-structure component amplification factor, which varies from 1.0 to 2.5. A value for a_p shall be selected from Table 15. Alternatively, this factor may be determined based on the dynamic properties of or empirical data on the component and the structure that supports it. The value shall not be taken as less than 1.0;
- R_p = Component response modification factor, which shall be taken from Table 15, except that R_p for anchorage shall equal 1.5 for shallow expansion anchor bolts, shallow chemical anchors, or shallow cast-in-place anchors. Shallow anchors are those with an embedment length-to-diameter ratio of less than 8. When anchorage is adhesive, R_p shall equal 1.0; and
- I_p = component importance factor; see Table 16.

Component	Description	a_p	R_p
Elements of the structure	Unbraced (cantilevered) parapets	2.5	3.0
	Exterior walls at or above the ground floor and parapets braced above their centers of gravity	1.0	3.0
	All interior bearing and nonbearing walls	1.0	3.0
Architectural components	Exterior and interior ornamentation and appendages	2.5	4.0
	Storage racks (including contents) over 1.8 m tall	2.5	2.5
	Permanent floor-supported cabinets and bookcases more than 1.8 m tall (including	1.0	3.0

Component	Description	a_p	R_p
Elements of the structure	Unbraced (cantilevered) parapets	2.5	3.0
	Exterior walls at or above the ground floor and parapets braced above their centers of gravity	1.0	3.0
	All interior bearing and nonbearing walls	1.0	3.0
Architectural	Exterior and interior ornamentation and appendages (contents)	2.5	4.0
	Anchorage and lateral bracing for suspended ceilings and light fixtures	1.0	3.0
	Access floor systems	1.0	3.0
	Masonry or concrete fences over 1.8 m tall	1.0	3.0
	Partitions	1.0	3.0
Mechanical and electrical equipment	Tanks and vessels (including contents) and their support systems	1.0	3.0
	Electrical, mechanical, and plumbing equipment, and associated conduit, ductwork, and piping	1.0	3.0
	Any flexible equipment laterally braced or anchored to the structural frame at a point below the equipment's center of mass	2.5	3.0
	Anchorage of emergency power supply systems and essential communications equipment; anchorage and support systems for battery racks and fuel tanks that are necessary to operate emergency equipment	1.0	3.0
	Temporary containers with flammable or hazardous materials	1.0	3.0
Other	Rigid components with ductile material and attachments	2.5	6.0
	Rigid components with nonductile material or attachments	1.0	2.5
	Flexible components with ductile material and attachments	1.0	1.5
	Flexible components with nonductile material or attachments.	2.5	9.0

Table 15. Nonstructural component amplification and response modification factors^{†††††}

Occupancy	Description	I_p
Hospitals	All components	1.50

Table 16. Nonstructural component importance factors^{†††††}

The design lateral forces determined by using Eq. 21 shall be distributed in proportion to the mass distribution of the element or component.

Forces determined by using Eq. 21 shall be used to design members and connections that transfer these forces to the seismic-resisting systems. The reliability/redundancy factor, ρ , may be taken as equal to 1.0.

Forces shall be applied in horizontal directions, which result in the most critical loadings for design.

9.1.2 Specifying Lateral Forces

Design specifications for equipment shall either specify the design lateral forces prescribed herein or reference these provisions.

9.1.3 Relative Motion of Equipment Attachments

For equipment in hospitals with an I_p of 1.50, as defined in Table 16, the lateral-force design shall consider the effects of relative motion of the points of attachment to the structure.

†††††††† FEMA 356 was used as reference to derive these values

†††††††† FEMA 356 was used as reference to derive these values

9.1.4 Alternative Designs

When an approved national standard or approved physical test data provide a basis for the earthquake-resistant design of a particular type of equipment or other nonstructural component, such a standard or data may be accepted as a basis for design of the item, with the following limitations:

- Provide minimum values for design of the anchorage and the members and connections that transfer forces to the seismic-resisting system.
- The lateral seismic force, F_p , and the overturning moment used in the design of the nonstructural component shall not be less than 80% of the values that would be obtained by using these provisions.

10. ASSESSMENT OF NON-BUILDING STRUCTURES

10.1 Definition

Non-building structures include all self-supporting structures other than buildings that carry gravity loads and resist the effects of earthquakes. Non-building structures shall be designed to provide the strength required to resist the displacements induced by the minimum lateral forces that are specified in this section. Designs shall conform to the applicable provisions of other sections of the National Code, as modified by the provisions in this chapter.

The two types of non-building structures considered in this document are walkways between adjacent structures, concrete canopies (awnings), and water towers and tanks. Some other nonstructural components such as electrical transmission towers or radio masts are typically governed by wind loading and thus not covered in this standard.

10.2 Criteria

The minimum design seismic forces prescribed in this section are at a level that produces displacements in a fixed-base, elastic model of the structure that are comparable to those expected of the real structure when it responds to ground motion. Reductions in these forces is permitted when the design of non-building structures provides sufficient strength and ductility consistent with the provisions specified herein for structures, to resist the effects of seismic ground motions as represented by these design forces.

10.3 Weight, W

The weight, W , for non-building structures shall include all dead loads and any additional permanent loads.

10.4 Period

The period for the non-building component shall be computed using the provisions of National Code or other recognized standards.

10.5 Response reduction factor

Response reduction factor (R) equal to the smaller value of 2.0 and the value defined in the National Code.

10.5.1 Lateral Force

LSP shall be used in design.

10.6 Rigid structures

Rigid structures (those with period T less than 0.06 sec) and their anchorage shall be designed for the lateral force obtained from Eq. 22.

$$\text{Eq. 22.} \quad V = 0.7(PGA)IW$$

Where:

- PGA denotes the peak ground acceleration from Chapter 4
- W is the weight of the unit
- I is the importance factor and equal to 1.5 for hospital buildings

The force V shall be distributed according to the distribution of mass and shall be assumed to act in any horizontal direction.

10.6.1 Tanks with Supported Bottoms

Flat-bottom tanks or other tanks with supported bottoms, founded at or below grade, shall be designed to resist the seismic forces that are calculated by using the procedures for rigid structures provided in this chapter. The design shall also consider the entire weight of the tank and its contents. Alternatively, such tanks may be designed by using the following procedure:

- A design basis prescribed for the particular type of tank by an approved national standard, if the seismic zones and occupancy categories conform to the provisions of the NBC or IS.

10.6.2 Walkways

Ground floor or elevated walkways are often used to connect two adjacent buildings. These walkways use either concrete or light steel roofing, can be one or two stories tall, and are supported by either concrete or masonry walls or columns. The walkways shall be assessed to determine their capacity to earthquake loading and the connection between the walkway and adjacent buildings, as well as the effect of additional seismic mass of walkway and the potential for pounding on adjacent buildings shall be investigated.

When the design of walkway is found inadequate, it shall be retrofitted using the procedures listed in this document for building structures. When the pounding is found unacceptable, seismic separation joints between the walkway and adjacent building shall be provided.

10.6.3 Canopies

Many hospital buildings in Nepal have concrete entrance canopies. These units are heavy and are susceptible to collapse during earthquakes, thus presenting a life-safety hazard and functional disruption. Canopies shall be assessed to determine if they have adequate capacity to resist earthquake accelerations. If the design is inadequate, they need to be retrofitted.

10.6.4 Other Non-Building Structures

Non-building structures that are not covered explicitly here shall be designed to resist design seismic forces that are not lower than those determined by using the provisions of the National Code, with the following additions and exceptions:

- The total design base shear determined in accordance with the provisions of the National Code shall not be less than that resulting from Eq. 23.

$$\text{Eq. 23.} \quad V = 0.56PGA * I * W$$

- The vertical distribution of the design seismic forces in structures that are covered by this section may be determined by using the provisions of the National Code.

11. RETROFITTING PRINCIPALS

11.1 Retrofitting philosophy

Before developing retrofitting strategy for a facility (i.e. building or a group of buildings including, non-building structures etc.) the risk posed by various components shall be evaluated holistically. It shall address all the elements of the facility including principal structures (building or non-building such as canopies, walkways, access ways, water tanks, etc.), other elements (e.g. facades, parapets, gables, etc.) and non-structural elements (e.g. false ceiling, mechanical and electrical services, etc.) which could cause life-safety hazard and/ or disruption of function of a facility.

Further to the above, high risk elements shall be addressed first so the potential risks could be mitigated.

For a seismic retrofit project to be successful, the engineer must consider the following:

- A well thought out system looking issues holistically,
- Understanding implications of the proposed intervention on the operation of the facility and interlinkages between different facilities. Note, hospital facilities involve complex operations
- Due consideration of the costs of retrofitting relative to new construction, ensuring value for money
- Buy-in on the outcome of the investigation by the facility management. Involve the facility management from the very beginning and understand the issues raised by them.
- Seismic design complying with the provisions of reliable and proven methods
- Seismic detailing that provide ductility and allow the structure to undergo the level of deformation anticipated by the designer

Construction that follows the structural plans and specifications and thus ensures the constructed product meets the intent of the engineer. The last bullet will be developed as part of construction quality assurance (QA).

11.2 General consideration

While developing retrofitting schemes for health facilities, the following shall be critically evaluated:

- Minimal intervention to the facility
- Minimal cost to bring the facility to expected resilience
- Minimal downtime, i.e., time required for implementation of the retrofitting scheme
- Minimal environmental disturbance (noise, dust, etc.)

11.3 Seismic retrofitting of buildings and non-building structures

While developing retrofitting schemes for health facilities, the following shall be critically evaluated:

- Reduce mass where possible
- Remove irregularities
- Improve integrity, important for URM buildings,

11.4 Retrofitting standard

Considering reduced useable life, the existing buildings, it is recommended to strength these buildings or their components to at least 75% of what is required for similar new buildings. What that means is any building component meeting 75% of the requirements set for a component of a new building is not required to be strengthened. However, this should be judiciously decided. For example, if all the components of a building meets say 90% requirement or more, then it does not make a sense to strengthen a few deficient components to just 75% of the demand.

12. SEISMIC RETROFITTING OF BUILDING STRUCTURES

12.1 Overview

Following the assessment of the buildings and other components as described in Chapter 7 through 10, if the facility is found to be non-compliant and expected response to be unacceptable, seismic retrofitting shall be implemented. . This chapter outlines general philosophy for seismic retrofitting. It also includes the seismic retrofitting option for concrete frame and masonry bearing wall buildings using RCSW. As discussed earlier, it should be noted that RCSW could be a reasonable system for RC frame with masonry infill, but for masonry buildings its applicability shall be evaluated carefully. This chapter outlines the seismic retrofitting option for concrete frame and masonry bearing wall buildings using RCSW. Additional retrofitting options applicable to specific deficiencies, and the deficiencies addressed by these measures, are presented in the Chapter 13.

12.2 Seismic retrofit of LFRS

12.2.1 General considerations

The seismically deficient buildings shall be modified by adding new reinforced concrete shear walls (RCSWs) in both lateral directions. The seismic retrofitting shall comply with all of the following requirements:

- While using RCSW for UE+RM buildings, the strength and stiffness of the existing masonry shall be accounted for.
- The new RCSWs shall be designed to carry 100% of the seismic loading or as relevant, particularly for RC frame buildings.
- The walls shall be designed and detailed according to the relevant national Standard or equivalent.
- The new RCSWs shall use concrete with a minimum compression strength as specified in the relevant national Standard and only deformed bars shall be used.
- The existing diaphragm shall be checked for strength, stiffness, and the design of existing collectors to be checked. Diaphragm elements including collectors to be added or retrofitted as necessary.
- The existing foundation shall be checked and retrofitted or additional footings to be added as necessary to resist the seismic loading from the new walls
- The connection between the new elements and the existing elements and floor slabs shall be designed for the transfer of seismic loading

For RC frame buildings the new elements are designed to carry 100% of the seismic loading, the lateral resistance of the existing members shall be ignored. The seismic mass of the existing members shall be included in analysis. The existing members shall be checked for deformation compatibility as outlined in this document.

Note the following:

- The walls must be placed symmetrically to avoid introducing torsion into the building.
- If possible, it is preferable to place the walls along the exterior of the building to maximize the torsional stiffness provided by the walls.
- As a minimum, one wall segment shall be added to each floor of the building, in each principal direction, and on each side of the center of mass of the floor.
- Detailing requirements:

- The minimum wall thickness shall be 150 mm and as a minimum one layer of #4 (12 mm).
- The spacing of reinforcement shall not be greater than the maximum value specified in the relevant national Standard or equivalent.
- Steel reinforcement ratio shall not be less than specified in the relevant national Standard or equivalent.
- The concrete mix and design shall be reviewed and approved by the engineer prior to construction.
- The placement of the reinforcement and the pour of concrete shall be supervised to ensure compliance with the construction documents
- The new walls shall be cast-in-place concrete and be solid walls without large openings.

12.2.2 Stiffness

The in-plane stiffness properties of the new shear walls are specified to account for concrete cracking as discussed in this document. The out-of-plane stiffness of the new walls shall not be considered.

12.2.3 Strength

The strength of the new walls shall be computed based on the provisions of relevant national standard or equivalent. In computing the strength, the effect of axial load, shear, and flexure shall be taken into account

12.2.4 Analysis

The Linear Static Procedure as discussed in this document shall be used to determine the demand on the new walls.

12.2.5 Acceptance criteria

For the new RCSW that comply with the requirements specified in this document, design actions shall be compared with design strengths; and *m*-factors shall be selected from Table 17.

Case	<i>m</i> -factors
Governed by flexure	3
Governed by shear	2.5

Table 17. Numerical acceptance criteria for new concrete shear walls §§§§§

12.3 Out-of-plane retrofit of masonry walls

12.3.1 General considerations

When assessment shows that the existing masonry infill walls (concrete frame buildings) or bearing walls (brick or stone masonry wall buildings) do not have adequate capacity to prevent out-of-plane failure, these walls shall be retrofitted.

The suggested slenderness ratio of Table 13 assumes that the existing wall-floor diaphragm connections are strong enough to carry the inertial forces from the floors to the masonry walls. This is unlikely to be the case for Nepal buildings, many of which likely have minimal or no such connections other than if the floor/ roof are constructed of reinforced concrete cast-in-place slab. Accordingly, walls not supported at the top will likely to respond as cantilevers (unsupported at the floors) and are unstable.

§§§§§§§§§ FEMA 356 was used as reference to derive these values

12.3.2 Types of anchors

New anchors shall be provided to prevent the out-of-plane failure of walls that meet the requirements of Table 13. Anchors shall be placed in drilled and grouted holes to provide adequate attachment to both the wall and the floor. Headed anchor bolts, anchor plates, bent reinforcement are examples of acceptable anchors

12.3.3 Strength

Tension (pull out) and shear strength of anchors shall be based on manufacturer data or verified by testing. The force acting on an anchor shall be computed from Eq. 24

$$\text{Eq. 24.} \quad F = 1.2 * C_a W_p$$

Where:

F is the force resisted by the anchor

C_a is the short spectral acceleration from Chapter 4 (the plateau of spectra)

W_p is the weight of the wall tributary to the anchor (equal to area tributary to the anchor times the unit weight of wall)

12.3.4 Acceptance criteria

For all anchor types, *m*-factor equals unity.

12.3.5 Slender walls

For slender walls that do not meet the requirements of Table 13, wall bracing shall be provided to reduce the wall slenderness. Then, wall anchorage as discussed earlier shall be provided.

12.4 Diaphragms

For diaphragms, three items shall be checked:

- Shear capacity of the diaphragm
- Chords and collectors
- Attachment of diaphragms to columns and walls

12.4.1 Concrete (rigid) diaphragms

When a concrete diaphragm is found to be inadequate, the following measures shall be taken:

- Increase the shear capacity of the diaphragm by means of adding topping slab and reinforcement. The effect of additional seismic mass shall be considered in analysis
- Add concrete beams to act as collectors and chords
- Provide anchorage to walls and columns
- Add new concrete shearwalls within the span of the existing LFRS bays

The strength, stiffness and acceptance criteria shall be based on the requirements of Section 7.2.7 of this document. When new reinforced concrete elements are added, they shall meet the minimum material property specifications of Section 12.2.1.

12.5 Wood diaphragms (masonry bearing wall buildings)

When a wood diaphragm is found to be inadequate, the following measures shall be taken:

- Provide steel diagonal cross bracing or plywood diaphragm panels to carry 100% of the inertial force to the vertical elements
- Add blocking and anchorage to walls and columns

12.6 Foundations

When an existing foundation is found as inadequate or when new concrete walls are added and the existing foundation has insufficient strength, new concrete foundations shall be added or the existing foundation shall be strengthened to resist seismic loading. It is recommended that a geotechnical engineer be consulted to provide bearing capacity for the project site.

13. SUPPLEMENTARY SEISMIC RETROFITTING SOLUTIONS

13.1 Summary

In addition to Section 12, this section presents additional retrofitting techniques for retrofit of hospital buildings in Nepal. The proposed retrofit options focus on the types of vulnerable construction that were identified for public hospitals in Nepal that follow. The engineers might decide to use the option of adding new elements or the option of retrofitting the existing deficient elements or a combination of the two. Table 18 summarizes the retrofit options for deficient vertical elements of the LFRS. Table 19 summarizes the retrofit options for deficient horizontal elements of the LFRS. These retrofit options are described in more detail in the sections. Appendix A provides drawings and details for a number of listed retrofit options.

LFRS	New elements	Retrofit of existing elements
Brick or stone wall	Add new walls or Add new reinforced shotcrete or Use fiber-reinforced structural plaster	Grout inject cracks
		Repoint mortar
		Add reinforcement
		Increase out-of-plane capacity
		If stone masonry wall, add through stones or equivalent to tie the wythes together
RC moment frame with infills	Add new walls	Increase size of beams or columns
		Improve member detailing

Table 18. Proposed retrofit matrix for vertical elements of LFRS

Floor/roof type	Option 1 New elements	Option 2 Retrofit of existing elements
RC slab	Add horizontal frame, bracing, or fiber reinforced polymer (FRP)	Reinforce the connection of the slab to vertical elements
		Reinforce collectors and chords

Table 19. Proposed retrofit matrix for horizontal elements of LFRS

13.2 Concrete frame buildings with brick or stone masonry infills

This section provides information on some effective retrofit measures and presents typical ductile details that have been used previously in the seismic retrofit of concrete structures. The key seismic deficiencies of the existing concrete buildings are summarized in Table 20. Seismic retrofit solutions are listed in the last column of the table.

Category	Seismic deficiency	Retrofit options
LFRS	Inadequate lateral strength	Add new concrete shear walls
		Shotcrete members
		Add new beams or columns
		Reduce seismic mass
		Seismically isolate the building
	Inadequate lateral stiffness	Concrete jacket the members
		Add new concrete walls
		Increase the size of walls

Category	Seismic deficiency	Retrofit options
		Increase size of beams and columns
Irregularity	Soft or weak story	Add strength or stiffness to story
		Add buckling restraint brace (BRB)
		Add supplementary energy dissipation (dampers)
	Torsional irregularity	Add balancing walls, or moment frames
		Add BRBs
		Add dampers
	Weak column-strong beam	Jacket columns
Captive columns	Saw cut partial height masonry walls Separate the stairways from the column	
Discontinues walls	Add walls at floor between columns	
	Add BRB at floor below walls	
Detailing	Lack confinement	Concrete jacket
	Short splices	Remove cover, repair splices by welding or other acceptable methods
		Add confinement
	Inadequate shear strength for walls	Shotcrete walls
	Low reinforcement walls	Add vertical reinforcement
	Low flexural strength	Add concrete column boundary elements
	Lack of confinement	Add FRP to walls
Weak beam-column joints	Jacket joints	
Infill walls	Out-of-plane failure of the walls	Provide anchorage for the infill walls Provide bracing for the walls
Architectural components	Partition walls or stairways not intended as part of LFRS act as structural members	Replace partition walls with light nonstructural walls, or saw cut sides of the walls Isolate the stairways from the floor slabs at one or both ends
Diaphragms	Inadequate shear capacity of RC floors	RP overlays
	Inadequate collector or chord	Add concrete beams
	Inadequate connection to wood roofs	Connect concrete walls and wood diaphragm with anchors
Foundation	Inadequate foundation strength	Enlarge the footings
		Add seismic isolation
	Lack of connection between walls or columns and footing	Provide anchorage

Table 20. Seismic deficiencies and retrofits for concrete frame buildings

The following rehabilitation measures may be effective in retrofitting reinforced concrete moment frames with infills:

13.2.1 Jacketing existing members

The new materials should be designed and constructed to act compositely with the existing concrete. Where reinforced concrete jackets are used, the design should provide detailing to enhance ductility. Considered component strength should not exceed any limiting strength of connections with adjacent components. Jackets should be designed to provide increased connection strength and improved continuity between adjacent components.

13.2.2 Modification of the element by material removal from the existing element

Examples include removing or separating the nonstructural components to eliminate the interference with LFRS.

13.2.3 Improvement of deficient existing reinforcement details

Removal of cover concrete to modify existing reinforcement details should avoid damage to the core concrete and the bond between existing reinforcement and the core concrete. New cover concrete should be designed and constructed to achieve composite action with the existing materials.

13.2.4 Changing the building system to reduce the demands on the existing element

Examples include addition of supplementary LFRS such as concrete shearwall and mass reduction.

13.3 Concrete shear wall buildings

This section provides information on some effective retrofit measures and presents typical ductile details that have been used previously in the seismic retrofit of concrete shear wall structures. The key seismic deficiencies of the existing buildings are summarized in Table 21. Seismic retrofit solutions are listed in the last column of the table.

Category	Seismic deficiency	Retrofit options
LFRS	Inadequate lateral strength	Add new concrete walls
		Shotcrete members to increase shear capacity
		Add FRP to the walls to increase shear capacity
		Add vertical boundary element columns at the ends of the walls
		Add base isolation
		Reduce seismic mass
Diaphragms	Inadequate connection to concrete floors	Dill and bond reinforcement between the slabs and walls
Foundation	Inadequate foundation strength	Enlarge the footings

Table 21. Seismic deficiencies and retrofits for reinforced concrete shear wall buildings

The following measures may be effective in rehabilitating reinforced shear walls. All of the rehabilitation measures require an evaluation of the wall foundation, diaphragms and connections between existing structural elements and any elements that are added for rehabilitation purposes.

13.3.1 Addition of wall boundary components

Addition of boundary components may be an effective measure in strengthening shear walls or wall segments that have insufficient flexural strength. These members may be either cast-in-place reinforced concrete components or steel sections. In both cases, proper connections should be made between the existing wall and the added components. The shear capacity of the rehabilitated wall should be reevaluated.

13.3.2 Increased shear strength of wall

Increasing the shear strength of the web of a shear wall by casting additional reinforced concrete adjacent to the wall web may be an effective rehabilitation measure. The new concrete should be at least 100 mm thick, and should contain horizontal and vertical reinforcement. The new concrete should be properly bonded to the existing web of the shear wall.

13.4 Unreinforced brick or stone masonry bearing wall buildings

This section provides information on some effective retrofit measures and presents typical ductile details that have been used previously in the seismic retrofit of masonry bearing wall structures. The key seismic deficiencies of the existing buildings are summarized in Table 22. Seismic retrofit solutions are listed in the last column of the table.

Category	Seismic deficiency	Retrofit options
LFRS	Inadequate lateral strength	Add new concrete walls
		Structural plaster
		RC shotcrete
		Add FRP to the walls
		Add vertical reinforcement to unreinforced walls
		Splint and bandage
	Add base isolation	
	Connection between walls and walls	Improve connection between walls (install stitches made of steel, structural plaster, steel flats or deep anchors)
Main walls	Out-of-plane toppling	Add internal or external secondary columns (buttresses)
		Install strongbacks
Diaphragms	Inadequate connection to concrete floors	Add steel angles connections
	Inadequate wood or metal diaphragm strength	Add steel cross bracing or plywood diaphragm panel or thin RC topping on the floor
	Inadequate connection to wood floors	Connect concrete walls and wood diaphragm with anchors
Foundation	Inadequate foundation strength	Enlarge the footings
Stone Wall	Delamination	Provide through stones or equivalent to tie the wythes
		Add internal or external secondary columns (buttresses)
Gables	Out-of-plane toppling	Remove and replace with or light materials
		Brace the exiting wall
Parapet walls	Out-of-plane toppling	Remove and replace with reinforced concrete or light materials
		Brace the exiting parapet wall

Table 22. Seismic deficiencies and retrofits for masonry bearing buildings

The following measures may be effective in rehabilitating reinforced masonry bearing walls. All of the rehabilitation measures require an evaluation of the wall foundation, diaphragms, and connections between existing structural elements and any elements that are added for rehabilitation purposes.

13.4.1 Out-of-plane anchorage

The out-of-plane failure of masonry bearing walls is one of the most common modes of failure in earthquakes. An effective way of mitigating this issue is to provide through bolt anchorage for the walls or buttresses, strongbacks, etc.

13.4.2 Increased shear strength of wall

See Table 22.

13.4.3 Diaphragm strengthening

For flexible wood or light metal gage diaphragms, the diaphragm capacity and connection to the masonry walls are often inadequate. Diagonal steel braces or plywood diaphragm panels provide an effective way of enhancing the diaphragm action. Steel anchorages can be used to attach the exiting flexible or concrete diaphragms to the masonry walls.

13.5 Concrete diaphragms

13.5.1 Retrofit measures

Two general alternatives may be effective in correcting deficiencies: either improve the strength and ductility, or reduce the demand. Providing additional reinforcement and encasement may be an effective measure to strengthen or improve individual components. Increasing the diaphragm thickness may also be effective, but the added weight may overload the footings and increase the seismic loads. Lowering

seismic demand by providing additional lateral-force-resisting elements may also be effective rehabilitation measures.

13.6 Concrete footings

The following strategies are effective in seismic retrofit of shallow concrete foundations :

13.6.1 Enlarging the existing footing

Enlarging the existing footing may be an effective rehabilitation measure. The enlarged footing may be considered to resist subsequent actions produced by the design loads, if adequate shear and moment transfer capacity are provided across the joint between the existing footing and the additions.

13.6.2 Providing tension tie-downs

Tension ties may be drilled and grouted into competent soils and anchored in the existing footing to resist uplift. Increased soil-bearing pressures produced by the ties should be checked against the acceptance criteria for the selected performance level, as specified in this document. Piles or drilled piers may also be effective in providing tension tie-downs of existing footings.

13.6.3 Providing pile supports for concrete footings or mat foundations

Adding new piles may be effective in providing support for existing concrete footings or mat foundations, if the pile locations and spacing are designed to avoid overstressing the existing foundations.

13.6.4 Adding new grade beams

This approach involves adding grade beams to tie existing footings together where poor soil exists; to provide fixity to column bases; and to distribute lateral loads between individual footings, pile caps, or foundation walls.

13.7 Nonstructural components

13.7.1 Deficiencies and retrofit solutions

Table 23 summarizes the retrofit options for deficient anchorage and/or bracing of nonstructural components.

Nonstructural element	Retrofit options
Heavy partition walls	Provide wall bracing and anchorage.
	Provide wall bracing and anchorage, and fiber reinforced polymer (FRP) partition walls.
	Remove and replace walls with lighter Sheetrock-type walls.
Ducts and piping	Provide support, bracing, and anchorage to the floors or walls.
Shelving	Provide bracing and anchorage to floors and/or walls.
Elevated TVs or monitors	Strap item to the mounts and bolt the mounts to the structure.
Mechanical and electrical equipment	Provide proper anchorage to the structure.
	Add spring isolation
Parapets	Provide bracing.
	Remove heavy parapets and replace with lightweight handrail.

Table 23. Seismic deficiencies and retrofit for nonstructural components

13.8 Non-building structures

13.8.1 Walkways

For walkways that have seismic deficiencies, the following retrofit measures are available:

- Walkways that are attached to buildings and cause pounding can be retrofitted by adding a seismic separation joint between the walkway and buildings.
- For walkways with heavy concrete roofs, consider replacing the roof with lighter material.
- For independent walkways with inadequate seismic capacity, add steel bracing to carry lateral loading or add seismic dampers.
- For elevated walkways between adjacent buildings without independent support, reinforce the connection between the walkway and one of the buildings, and provide sliding joint with the other building.

13.8.2 Canopies

For canopies that do not have adequate capacity to resist earthquakes, the following upgrade options are available:

- Remove the canopy and/or replace it with lightweight construction.
- Provide an independent gravity and lateral support system (new moment frames, for example) and isolate the canopy from the building with an appropriate seismic gap.
- Provide a supplemental gravity support system near the outside edge of the canopy (new beams and/or columns) to eliminate or reduce the cantilever.
- Retrofit the deficient elements and connections to the building such that seismic force can be transferred to and resisted by the building.

13.8.3 Water towers and steel supported water tanks

- Strengthen members, connections, and anchorage
- Add bracing for tall members
- Add seismic isolation

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APPENDIX A SEISMIC RETROFIT DETAILING

A.1 General

This chapter presents examples of seismic detailing for structural and nonstructural components previously implemented by the authors of the standard the walls

A.2 Examples of detailing for seismic retrofitting of structural components

The following section provides examples of seismic retrofit detailing that has been used successfully.

A.3 Building types B1-B5 Masonry bearing walls

Figure A.1 presents the seismic retrofit of a deficient building with new reinforced concrete shear walls, designed to carry appropriate level of seismic loading in both directions. Note the symmetric placement and redundancy of walls.

The methods presented here for stone masonry buildings are equally applicable to brick masonry buildings with similar characteristics other than installation of through stones to mitigate delamination of stones walls.

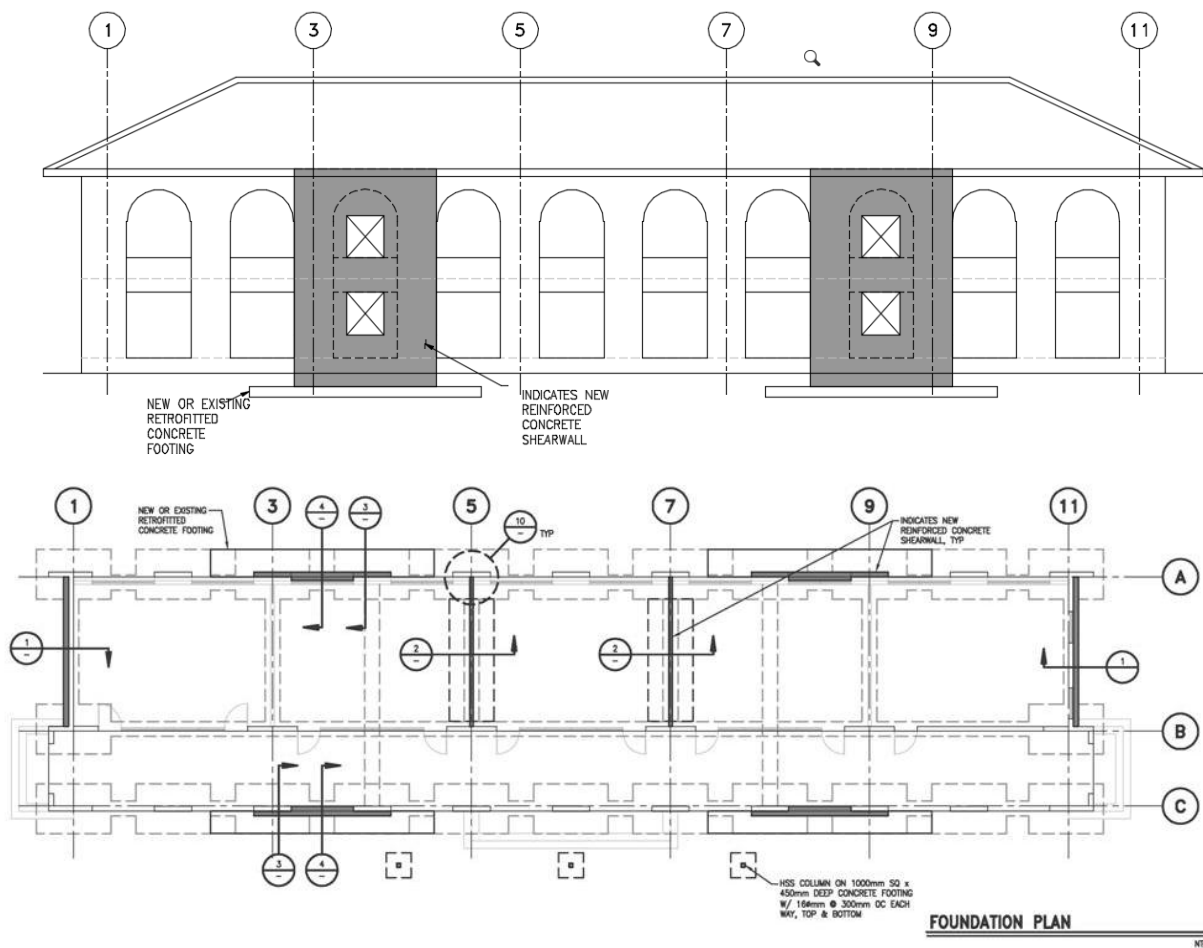


Figure A.1. Schematic plans and elevations of added concrete shear walls for seismic retrofitting

Figure A.2 presents an example of seismic retrofitting to prevent out-of-plane failure of masonry walls. Such retrofitting is applicable to both infill walls of moment frame buildings and walls of bearing wall structures.

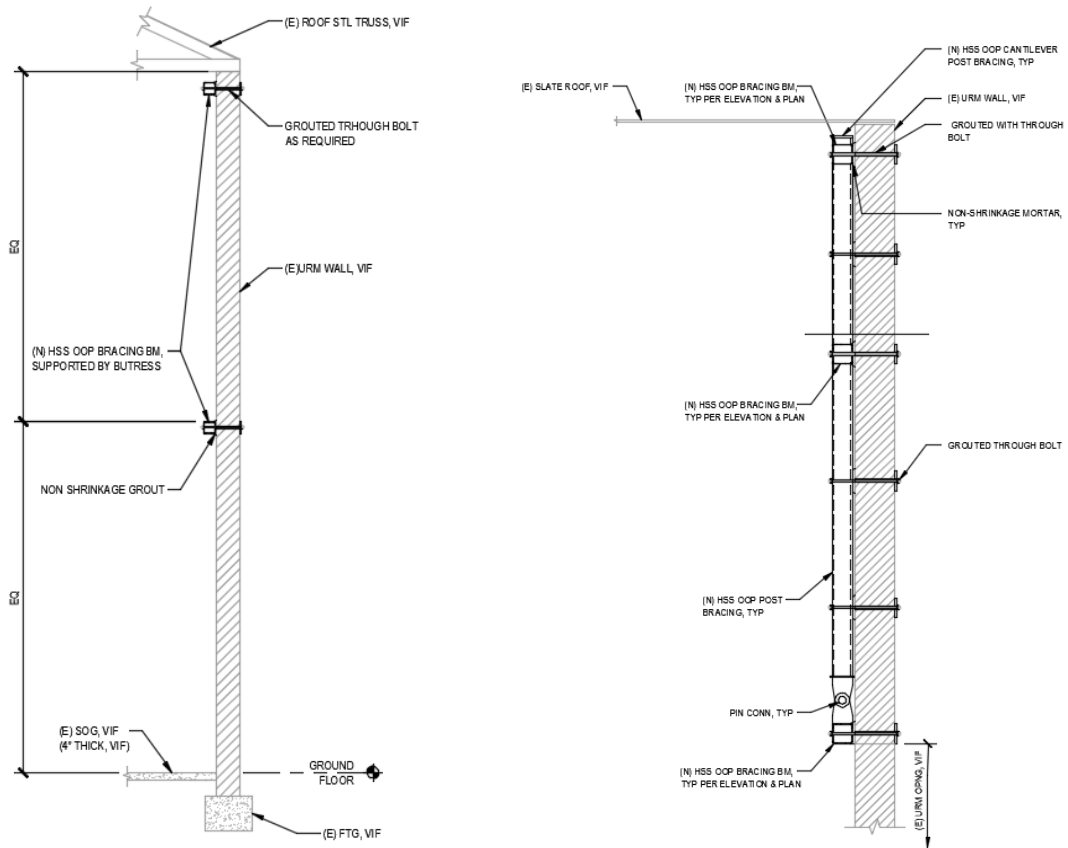


Figure A.2. Out-of-plane strengthening of wall connection detail

Figure A.3 through Figure A.19 present example of seismic retrofitting for the masonry bearing wall (Types B1 through B5) buildings.

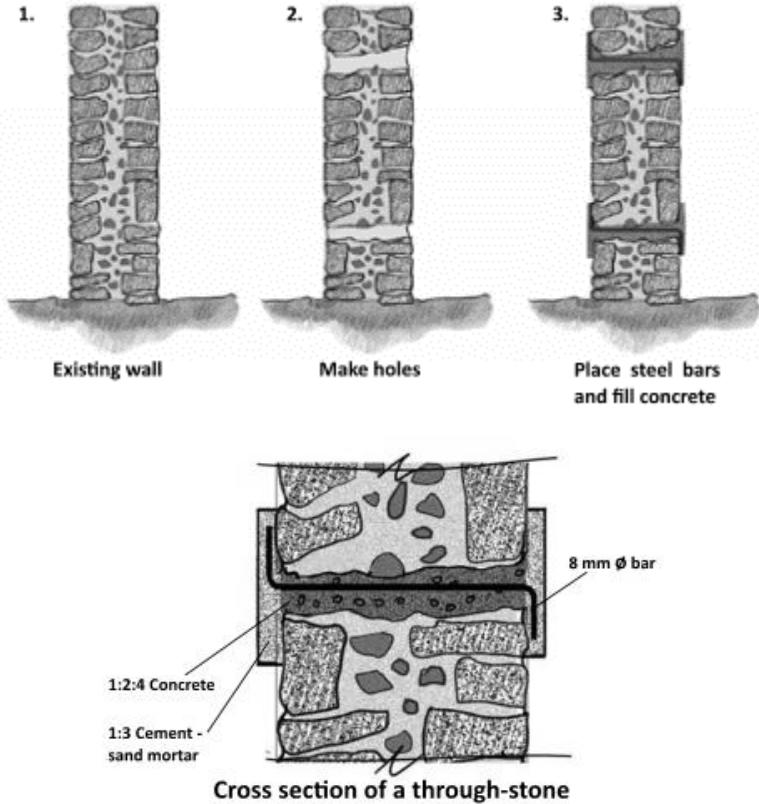


Figure A.3. Through-stones to prevent delamination (Bothara & Brzev, 2011)

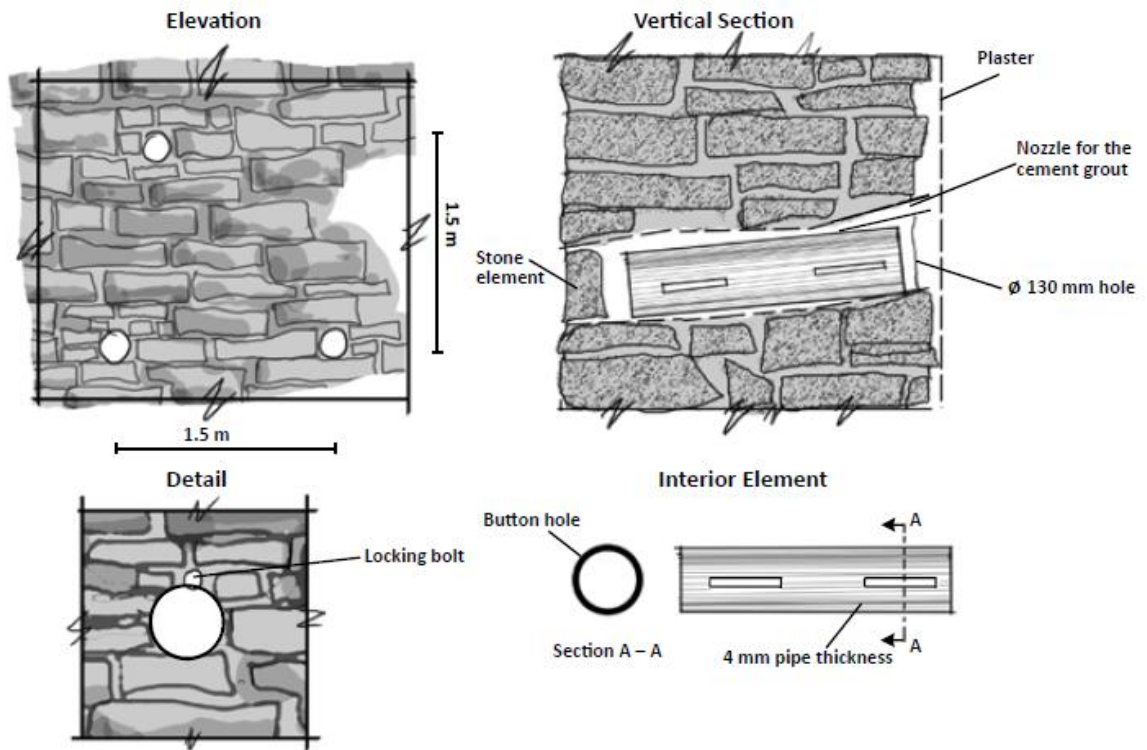


Figure 4.21 Installation of through-wall anchors in stone masonry walls after the 2002 Molise, Italy, earthquake (source: Maffei et al. 2006)

Figure A.4. Installation of through-wall anchors in stone masonry walls after the 2002 Molise, Italy, earthquake (Bothara & Brzev, 2011)

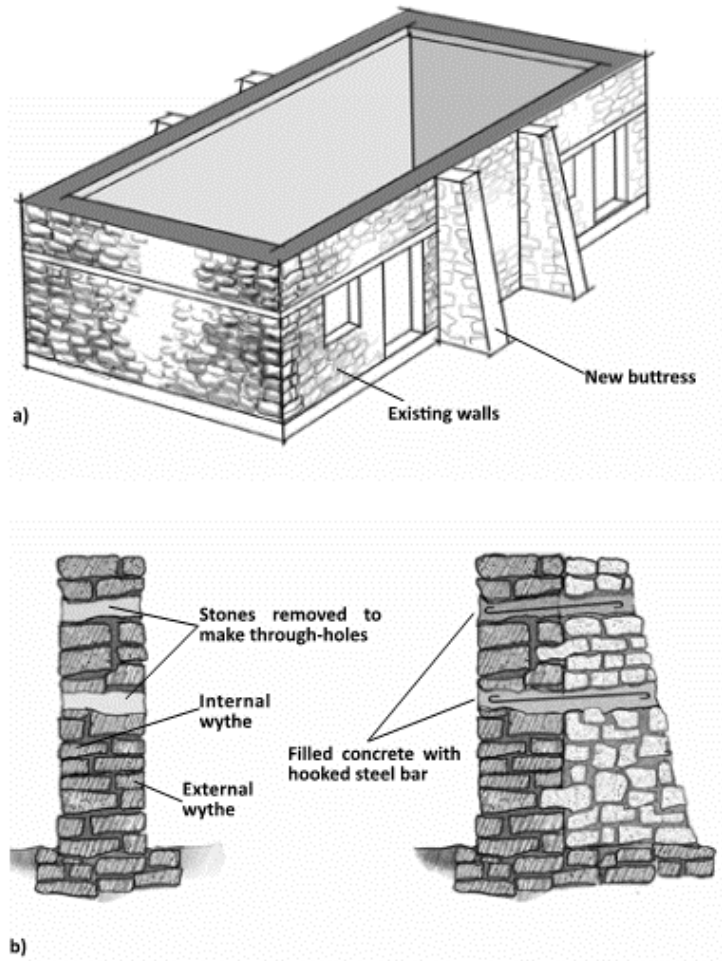


Figure A.5. Buttrass secondary frame (Bothara & Brzev, 2011)

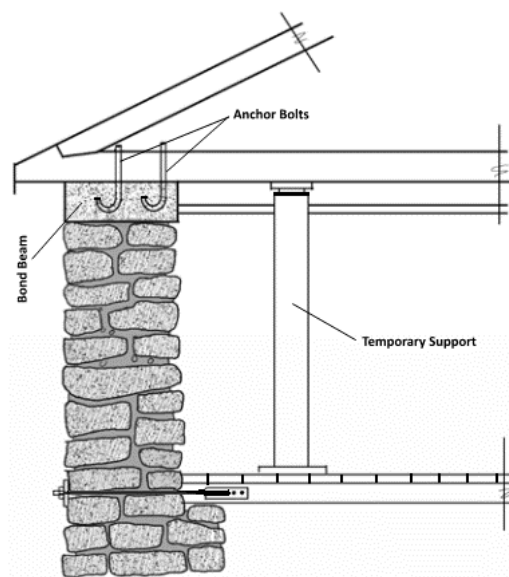


Figure A.6. New concrete band at the top to ensure “box-like” behaviour (Bothara & Brzev, 2011)

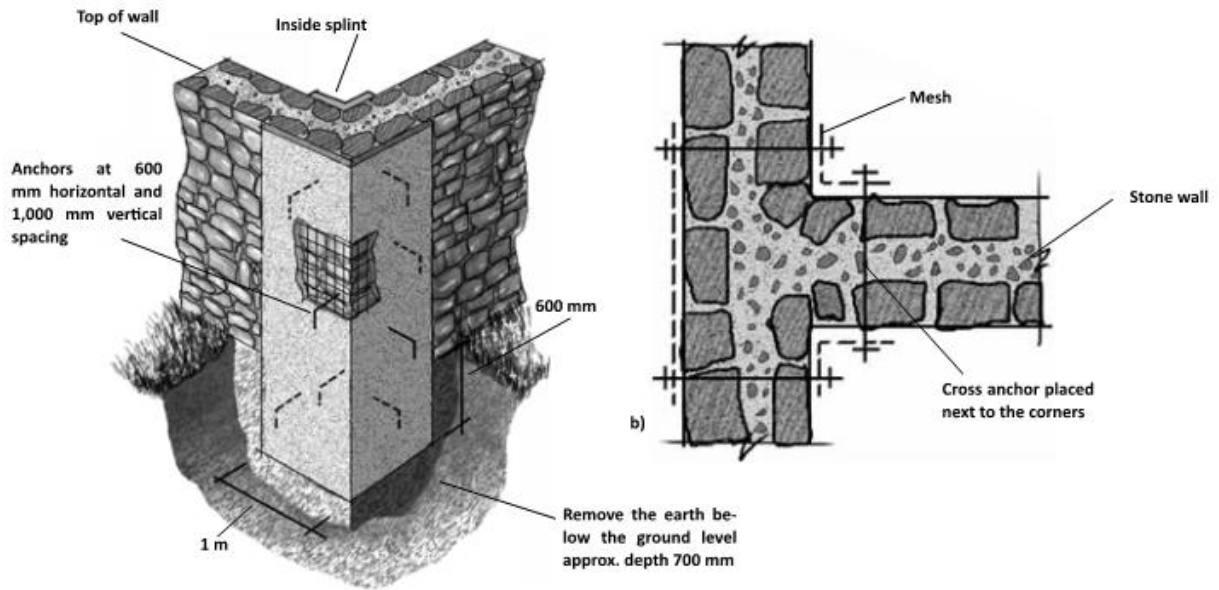


Figure A.7. Ferro cement splint to enhance the connections wall-to-wall (Bothara & Brzev, 2011).

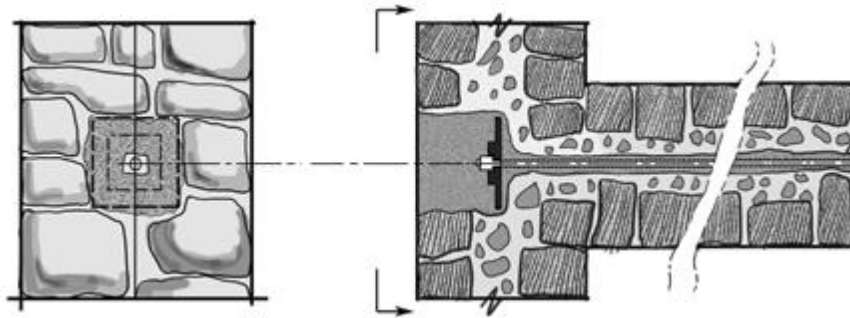
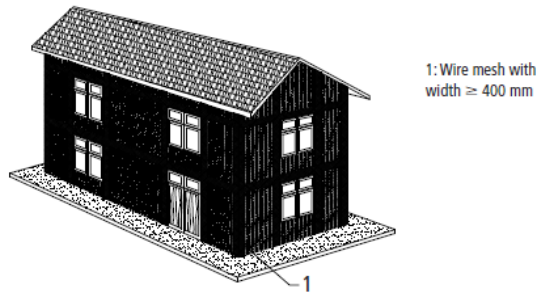


Figure A.8. Installation of post-tensioned steel anchors to enhance connections between the intersecting walls (Bothara & Brzev, 2011)



Schematic of splint and bandage (Arya, Boen & Ihiyama, 2013)

A school building strengthened with splint and bandage (Bothara et al, 2018)

Figure A.9. Splint and bandage

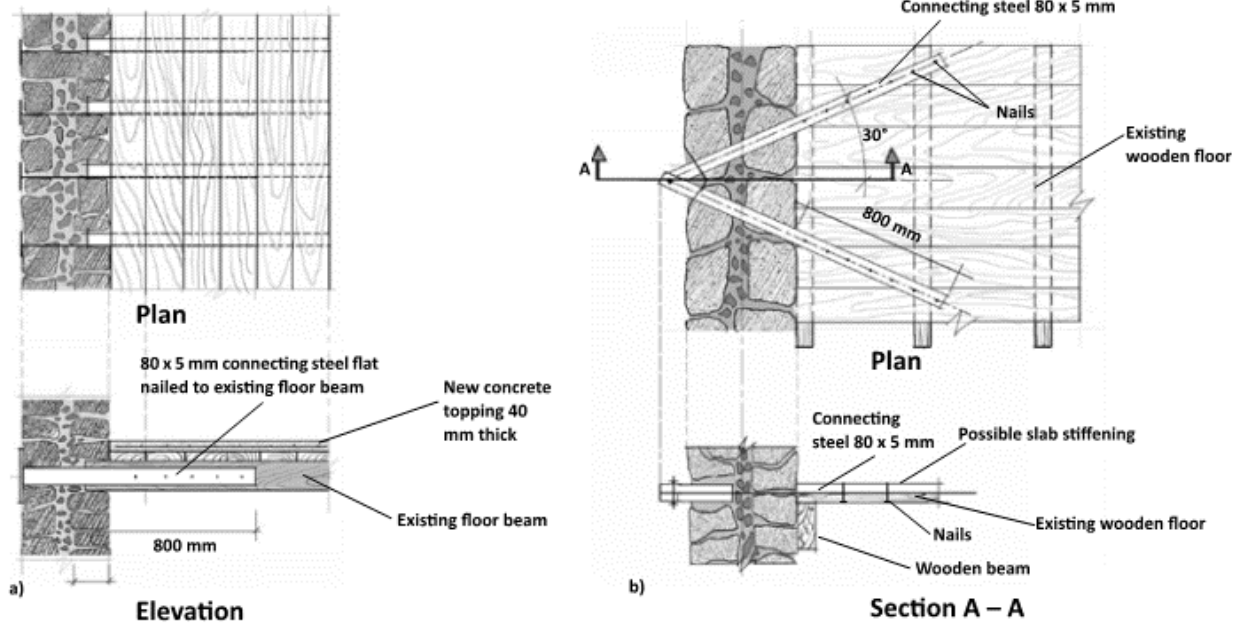


Figure A.10. Steel straps for wall-to floor anchorage: a) floor beams perpendicular to the wall, b) floor beams parallel to the wall (Bothara & Brzev, 2011)

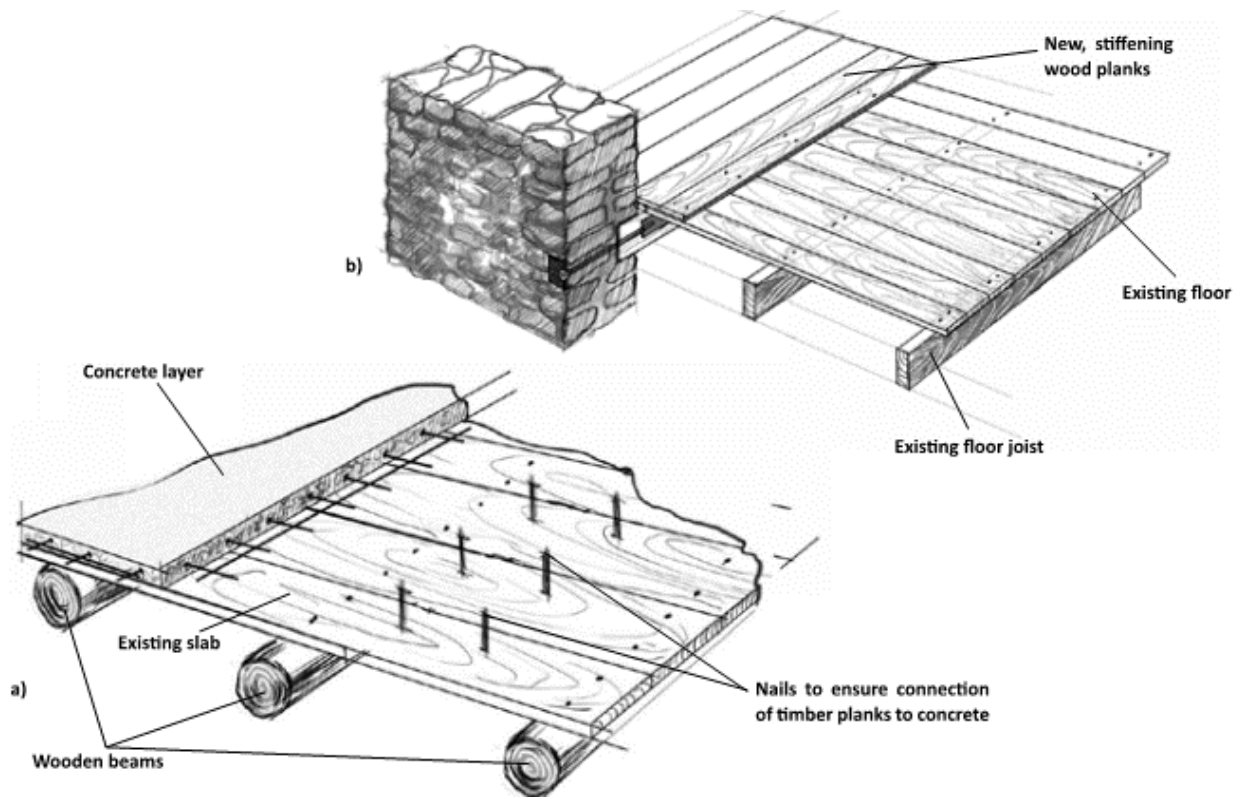


Figure A.11. Stiffening of the floor diaphragm by: a) thin RC topping, b) timber planks (Bothara & Brzev, 2011).

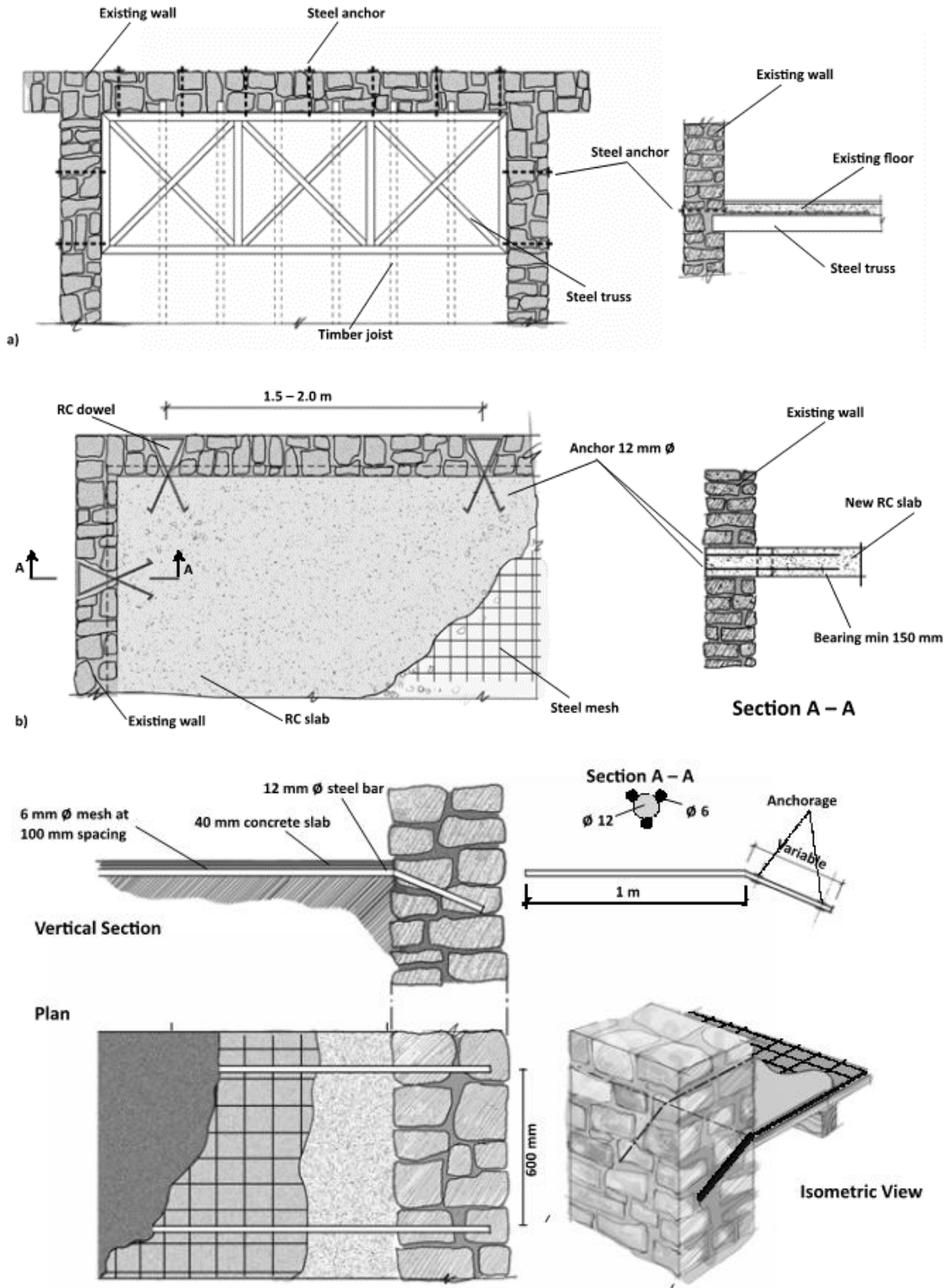


Figure A.12. Retrofitting the wall ensuring adequate connections to existing walls: a) diagonal braces, b) new RC slab (Bothara & Brzev, 2011)

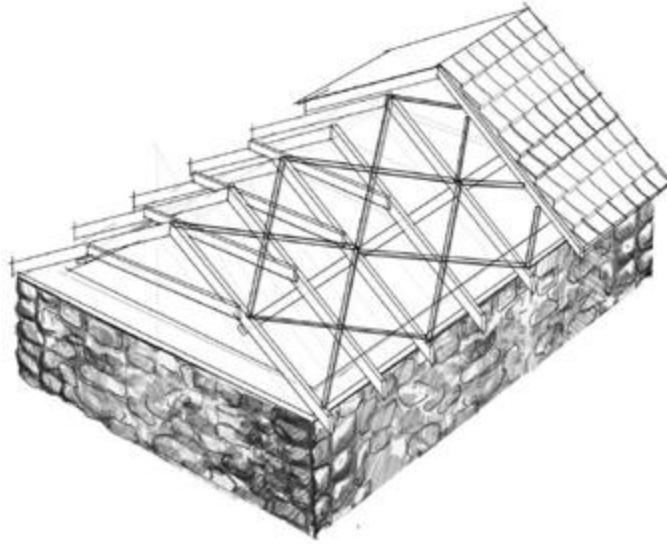


Figure A.13. Roof bracing (Bothara & Brzev, 2011)



Figure A.14. Lightly reinforced jacketing of a stone masonry wall in Slovenia (Bothara & Brzev, 2011)

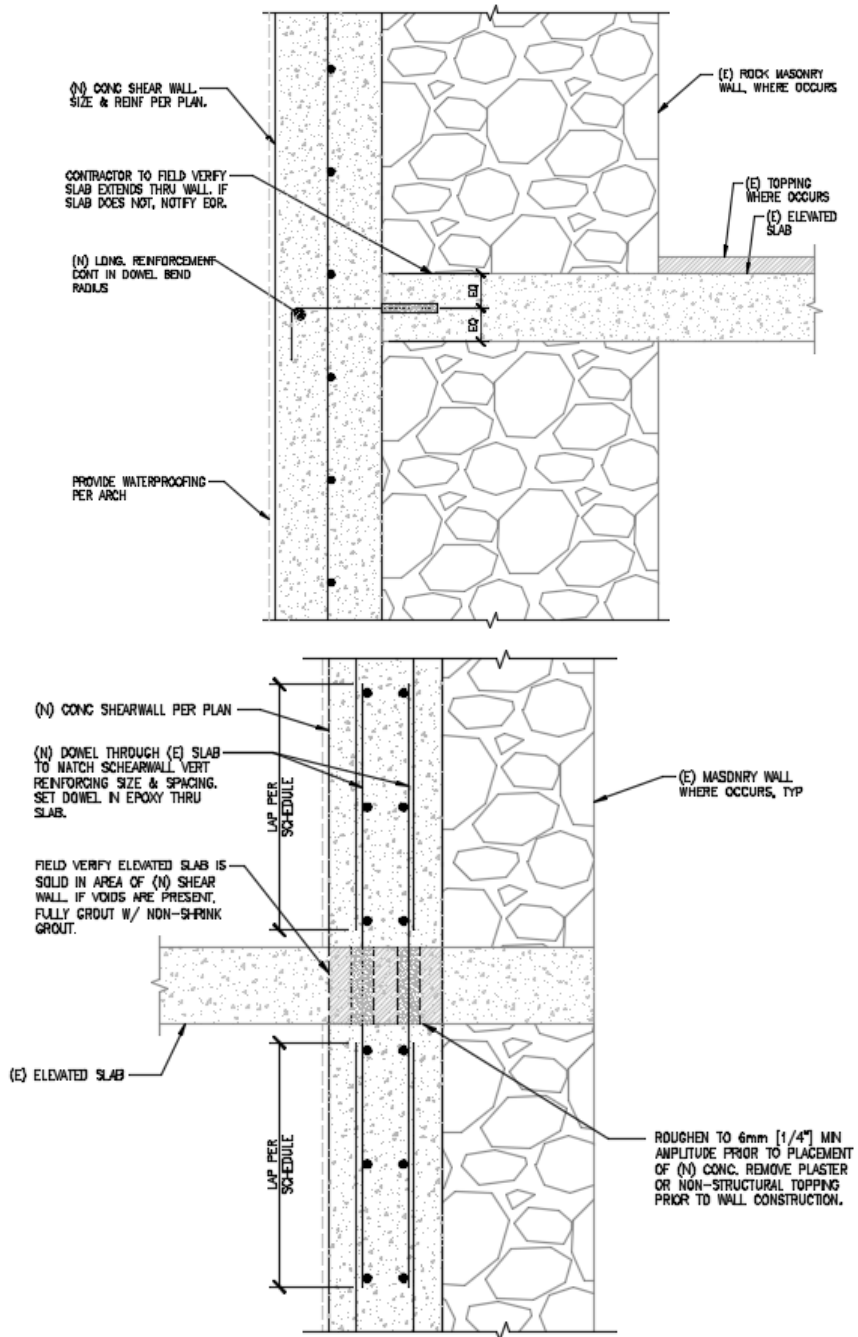


Figure A.15. Add connection of wall with existing masonry

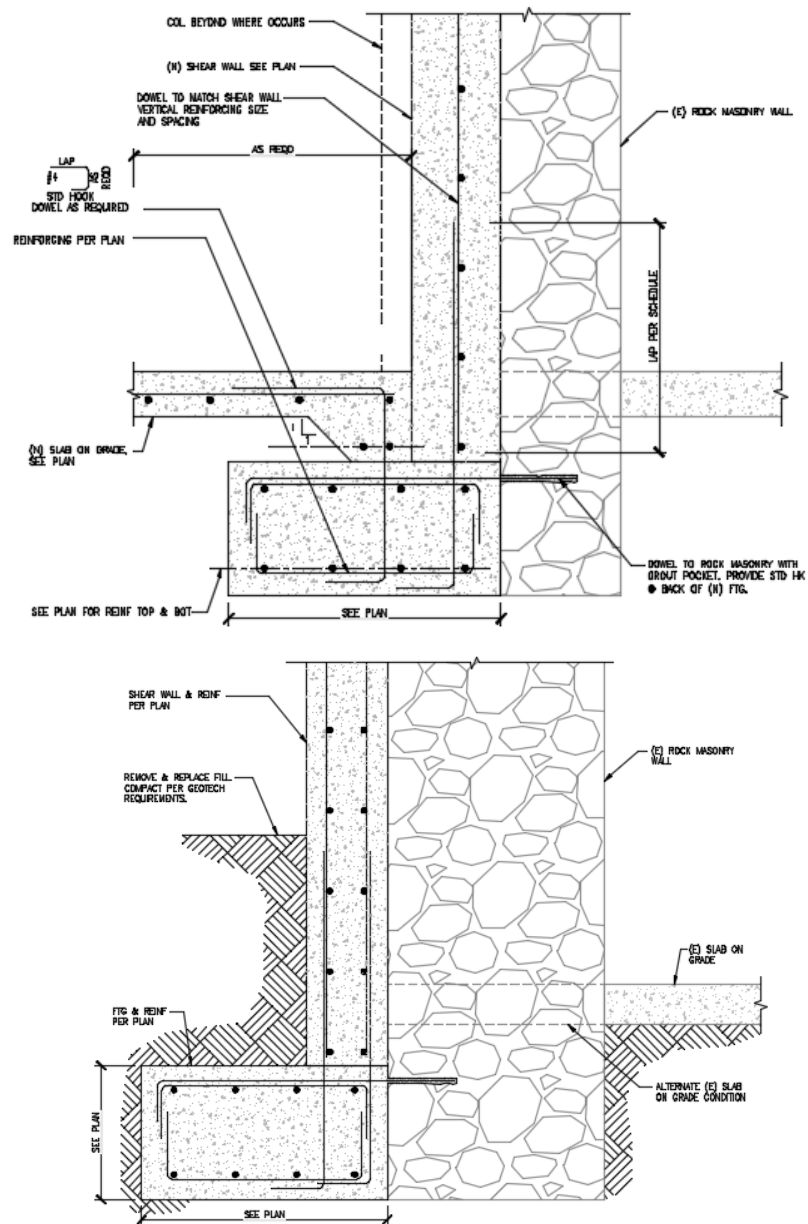


Figure A.16. Add ductile footing and concrete shear wall and connect with Stone Masonry Wall

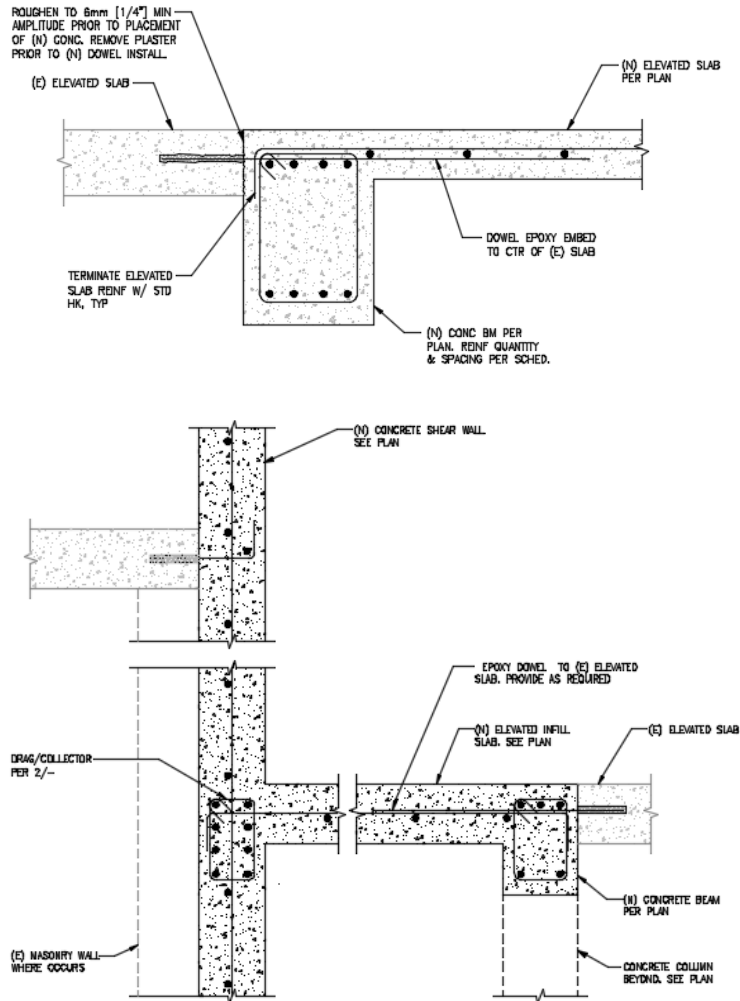


Figure A.17. Add connection of beam/shear wall with existing slab

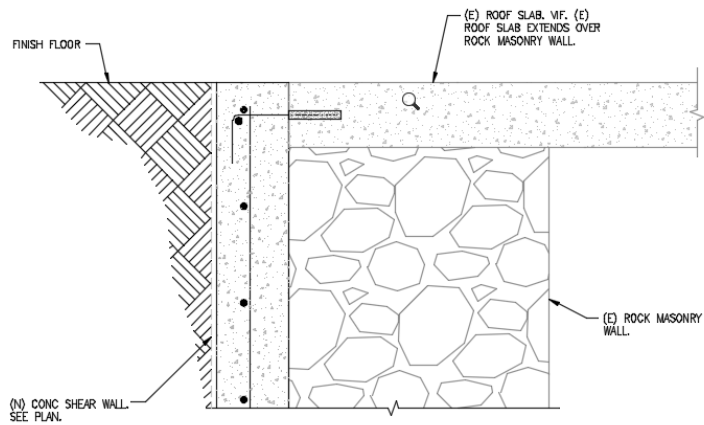


Figure A.18. Add connection of concrete beam/shearwall with existing slabs and beams



Figure A.19. Parapet Bracing (Marco Panichi)

A.4 Building types B6-B7 reinforced concrete moment frame buildings

Figure A.20 presents the seismic retrofitting of a concrete moment frame building by the addition of new reinforced concrete shear walls.

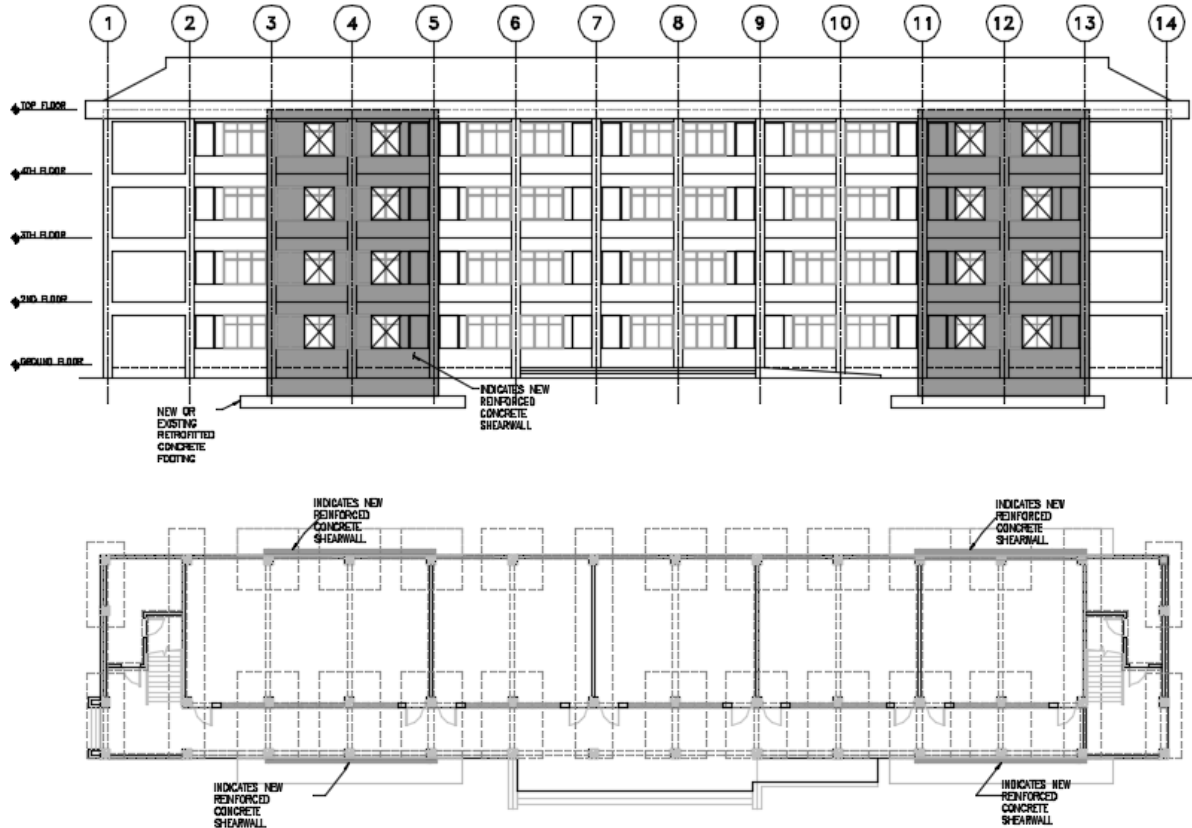


Figure A.20. Seismic retrofitting by addition of concrete shear walls

Figure A.21 through Figure A.35 present examples of seismic retrofit details for concrete buildings. The more conventional retrofit techniques of adding new walls or mitigating non-ductile members are presented first. The innovative approaches of use of BRB, braced frames, seismic dampers, and base isolation details are also shown.

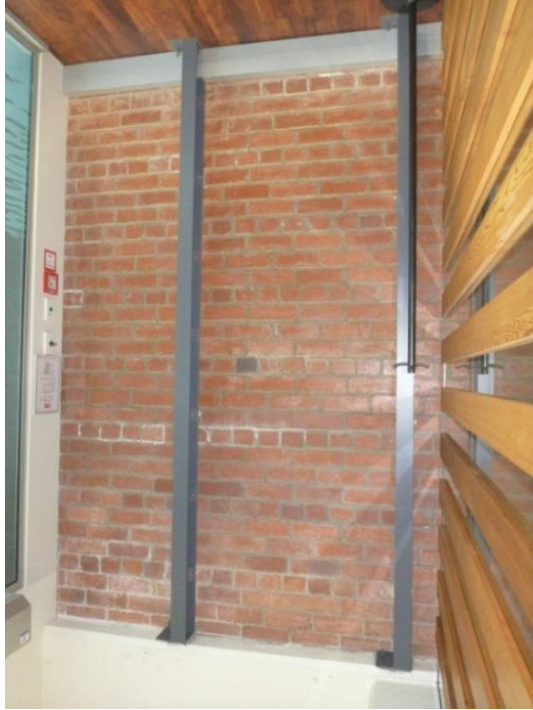


Figure A.21. Bracing of wall against face load (MBIE, 2017)

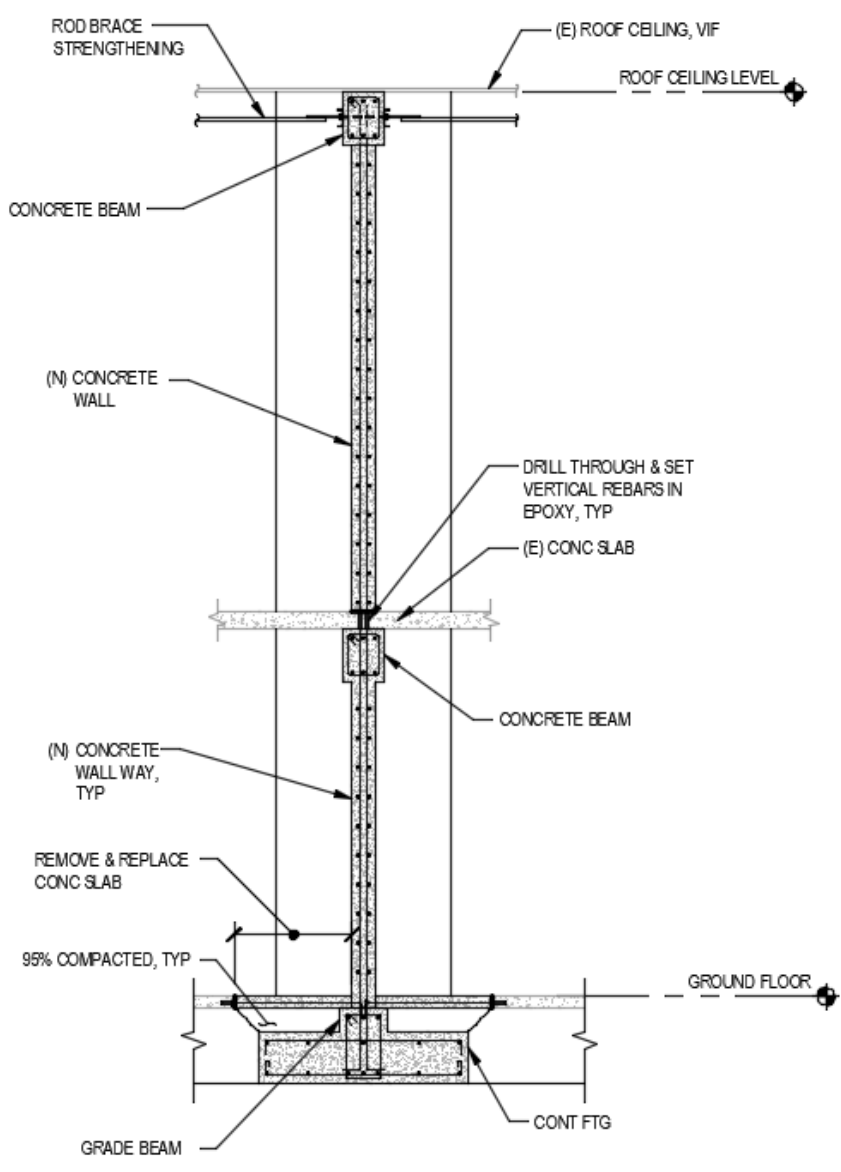
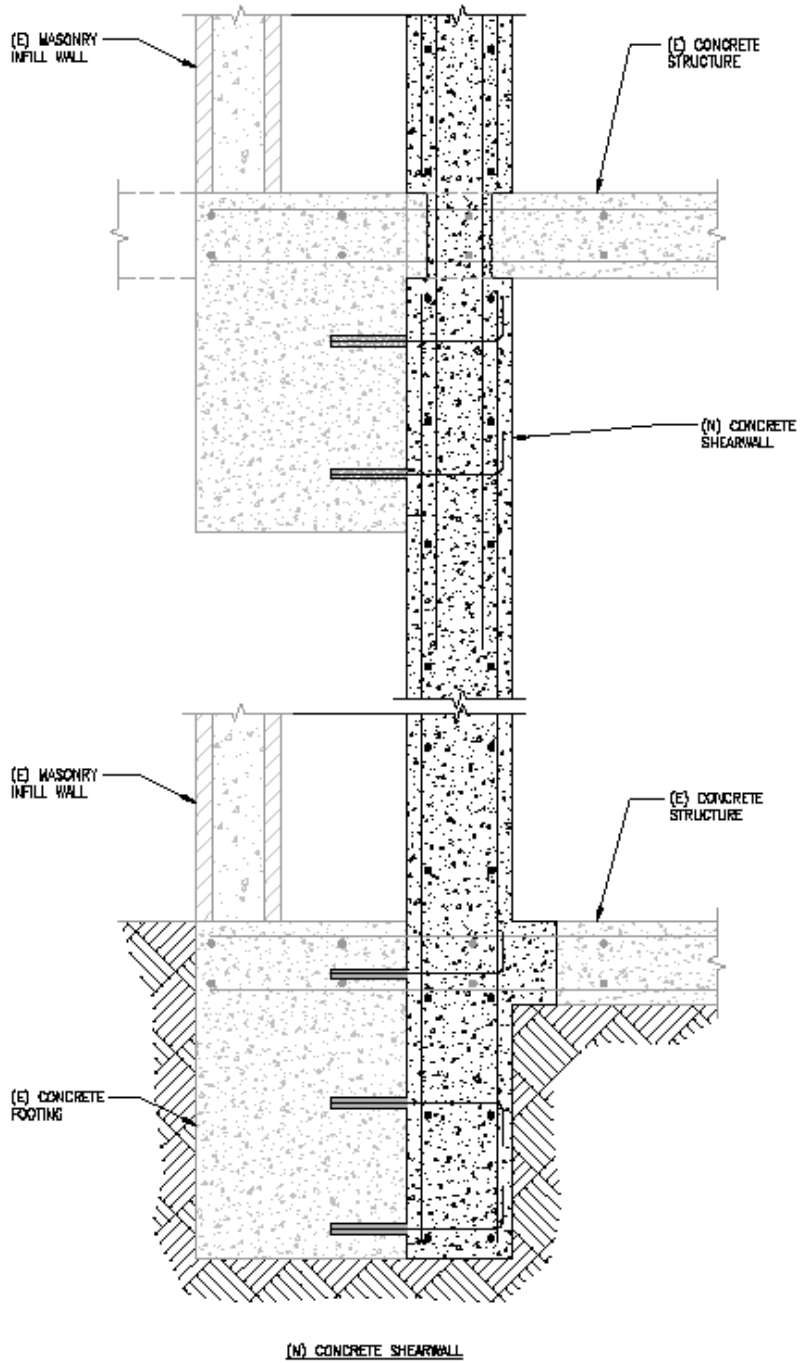


Figure A.22. New internal wall retrofits



SECTION DETAIL

Figure A.23. Add new ductile concrete wall

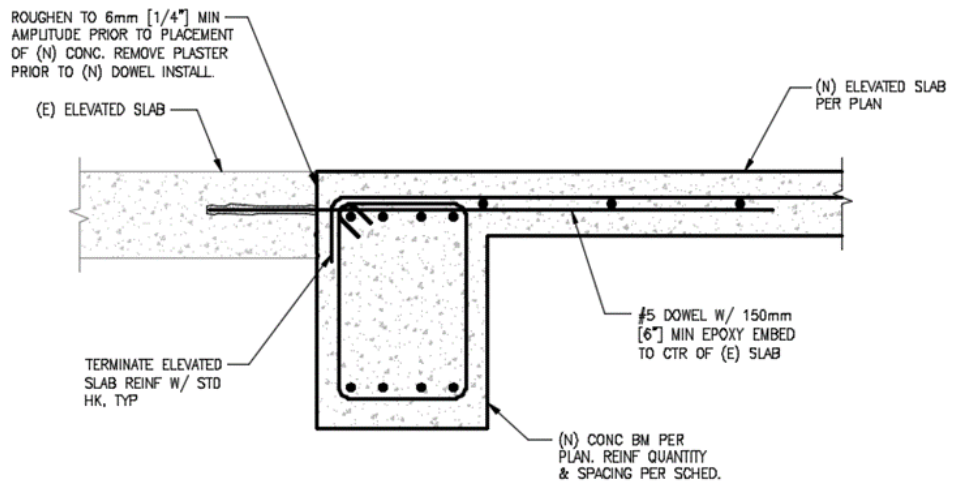


Figure A.24. Add connection of beam/shear wall with existing slab

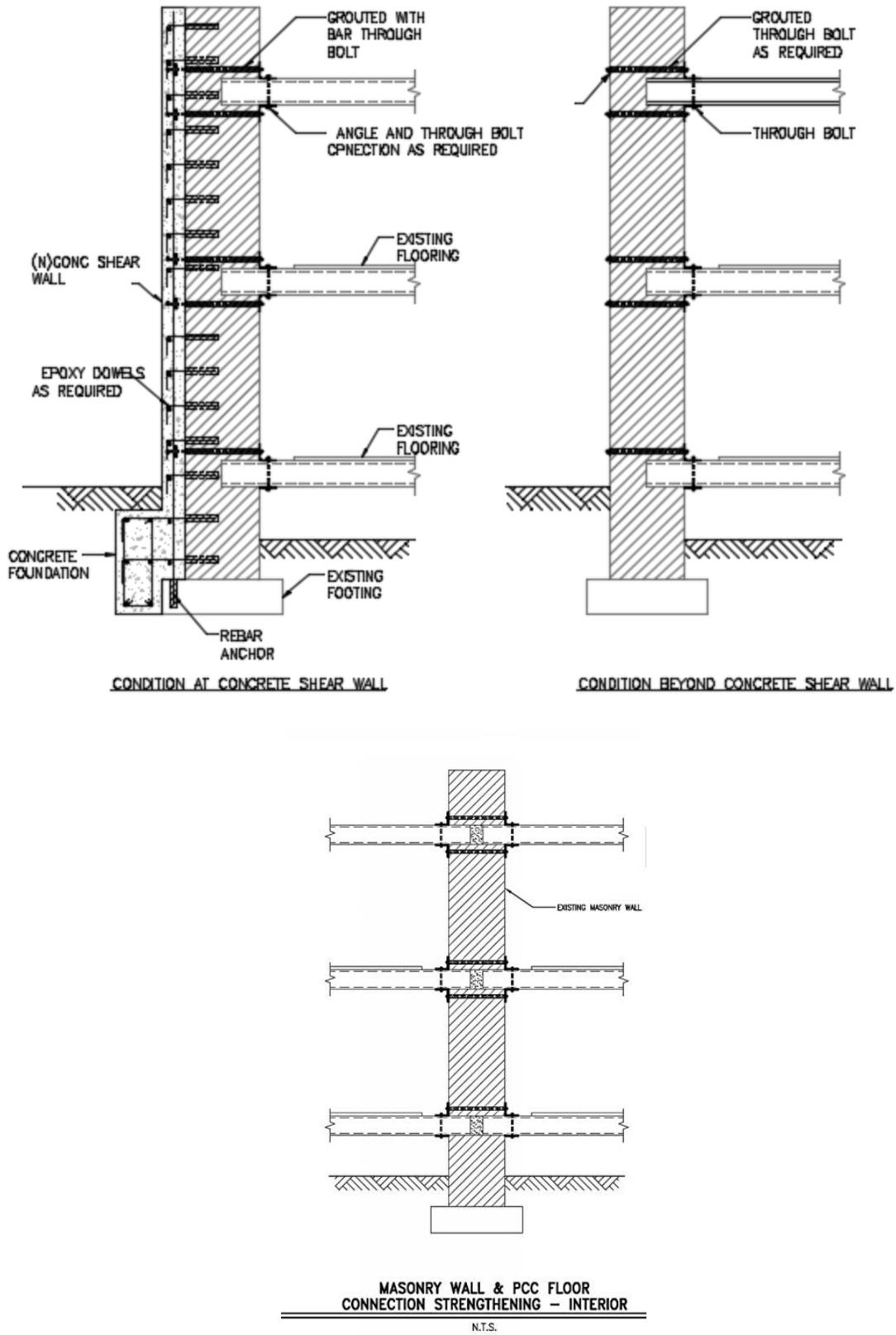


Figure A.25. Strengthening of existing walls with RC Shear walls, connection details

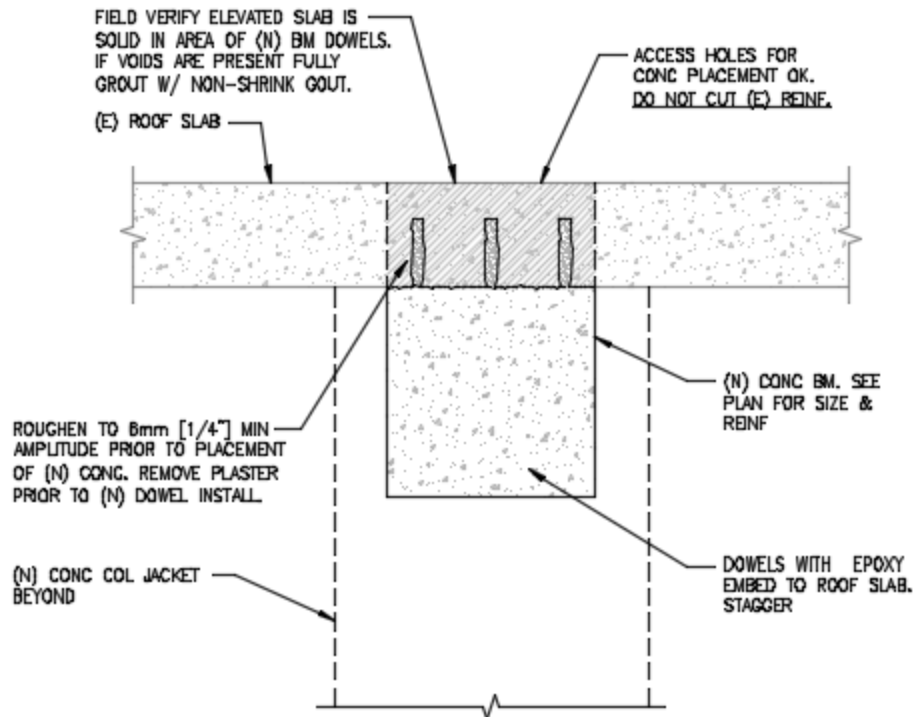


Figure A.26. Add connection of concrete beam/shearwall with existing slabs and beams

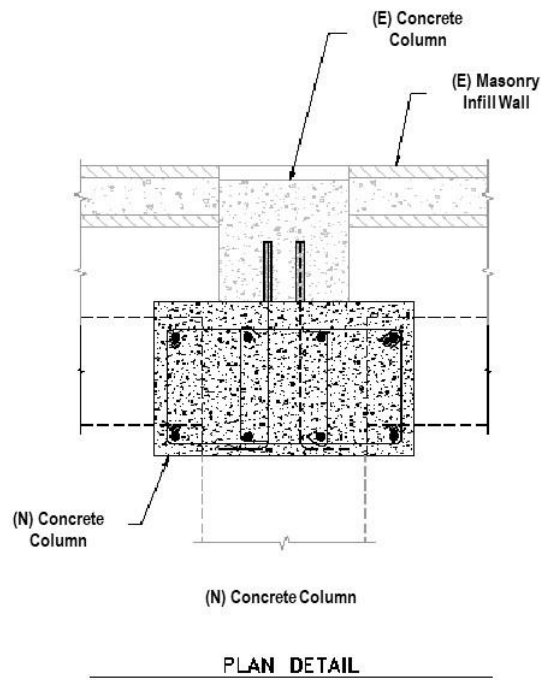


Figure A.27. Add new ductile concrete column adjacent to existing nonductile concrete

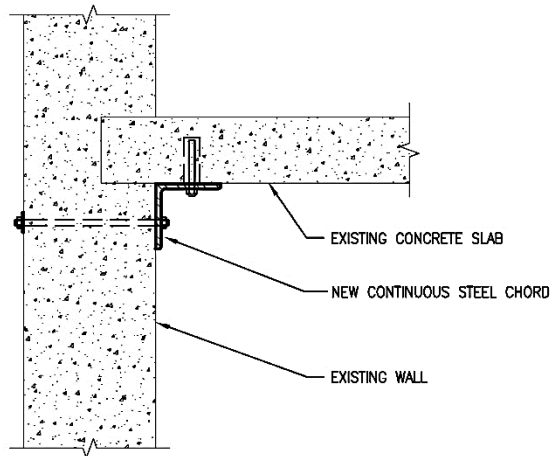


Figure A.28. Strengthening of slab-to-wall connection

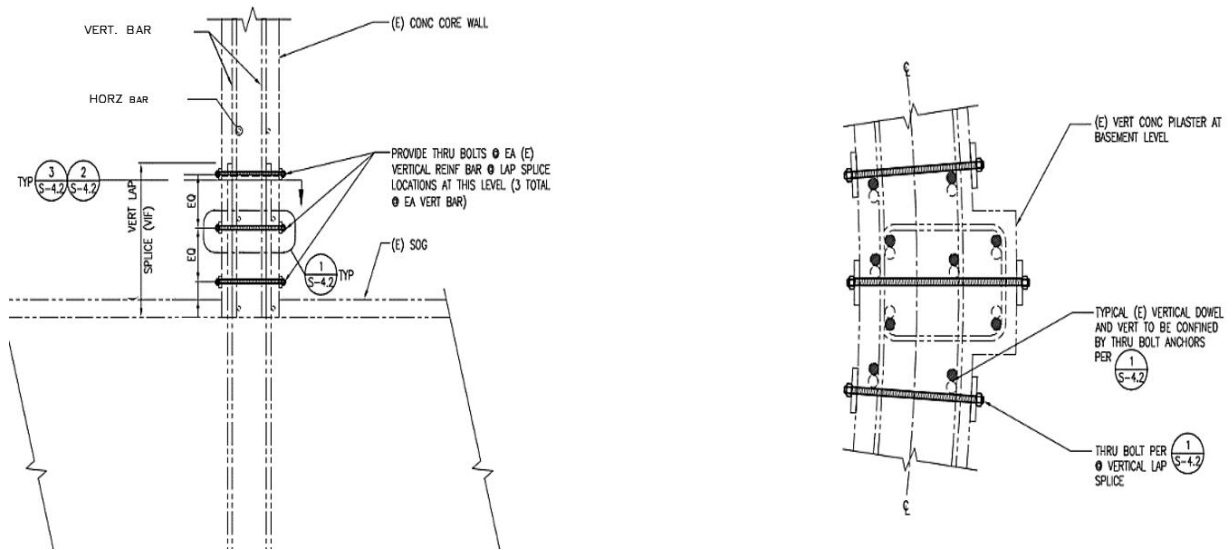


Figure A.29. Repair of short lap splices by added confinement

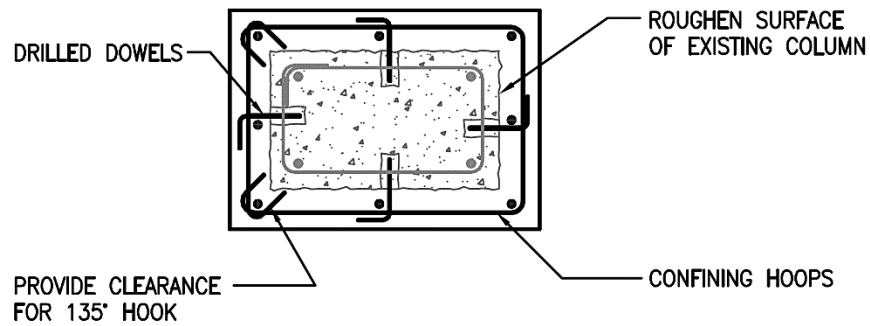


Figure A.30. Concrete jacking of concrete members

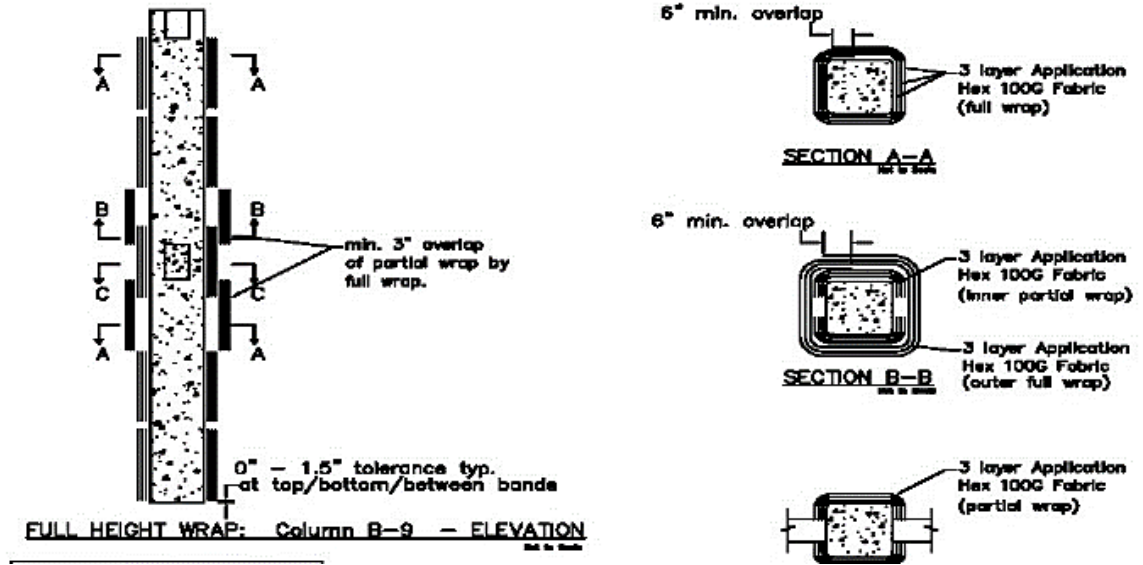


Figure A.31. FRP jacketing of members

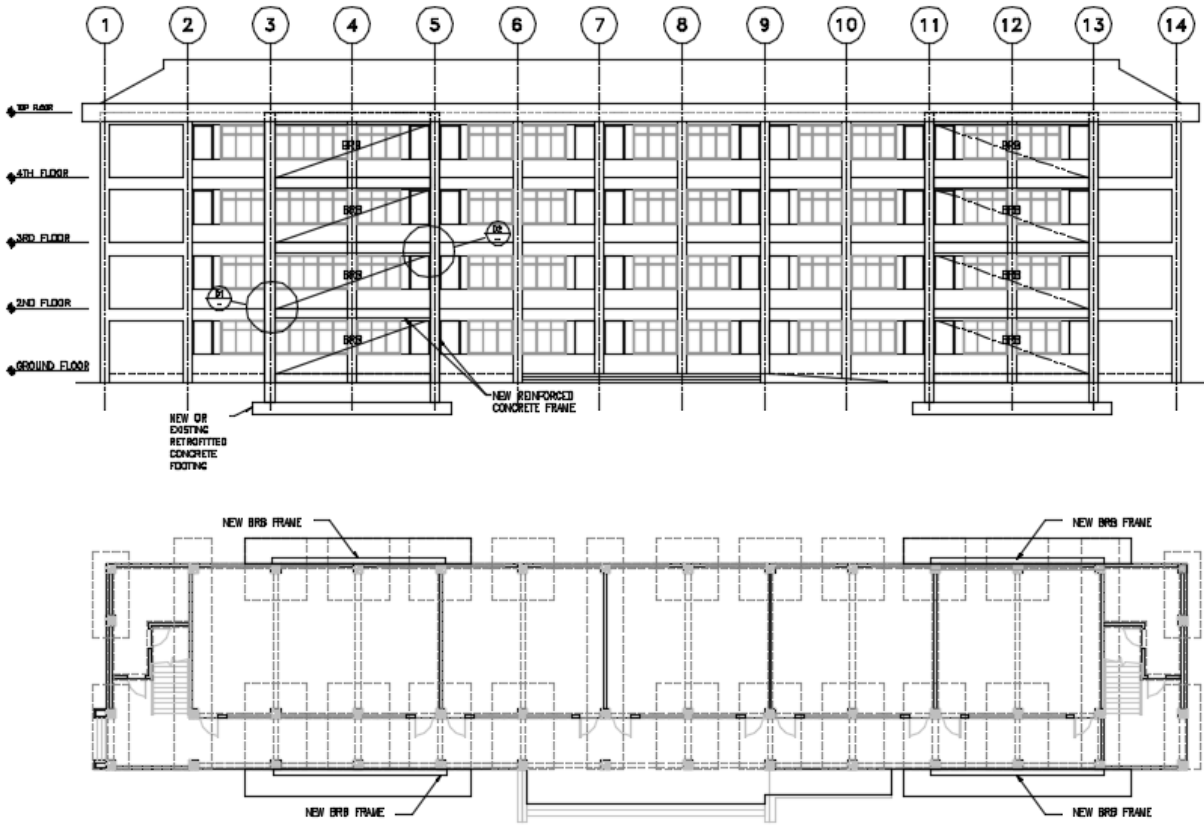


Figure A.32. Seismic retrofitting by adding BRBs



Figure A.33. Strengthening of RC frame building with concentrically braced frame

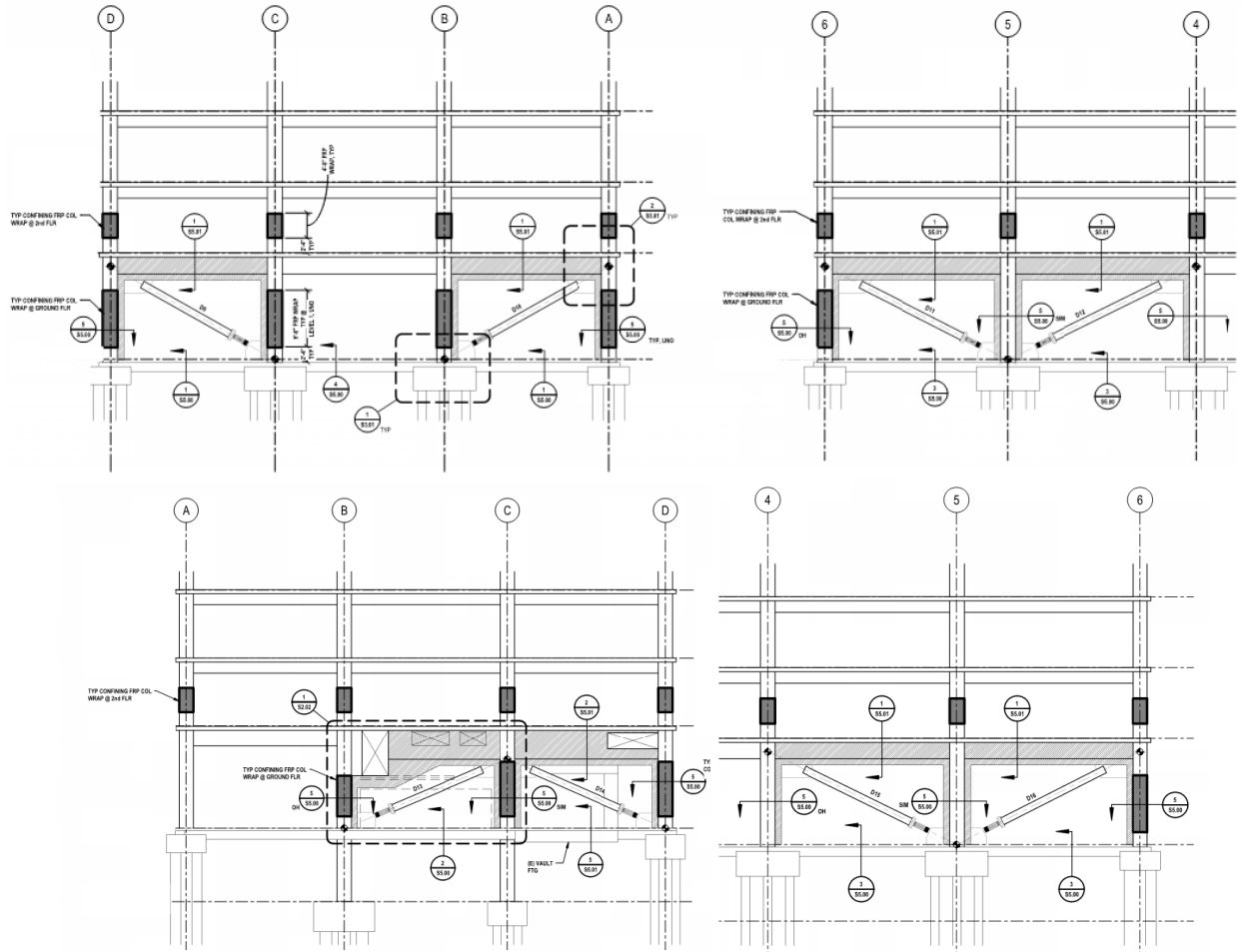


Figure A.34. Retrofit of seismic irregularities by the addition of seismic dampers

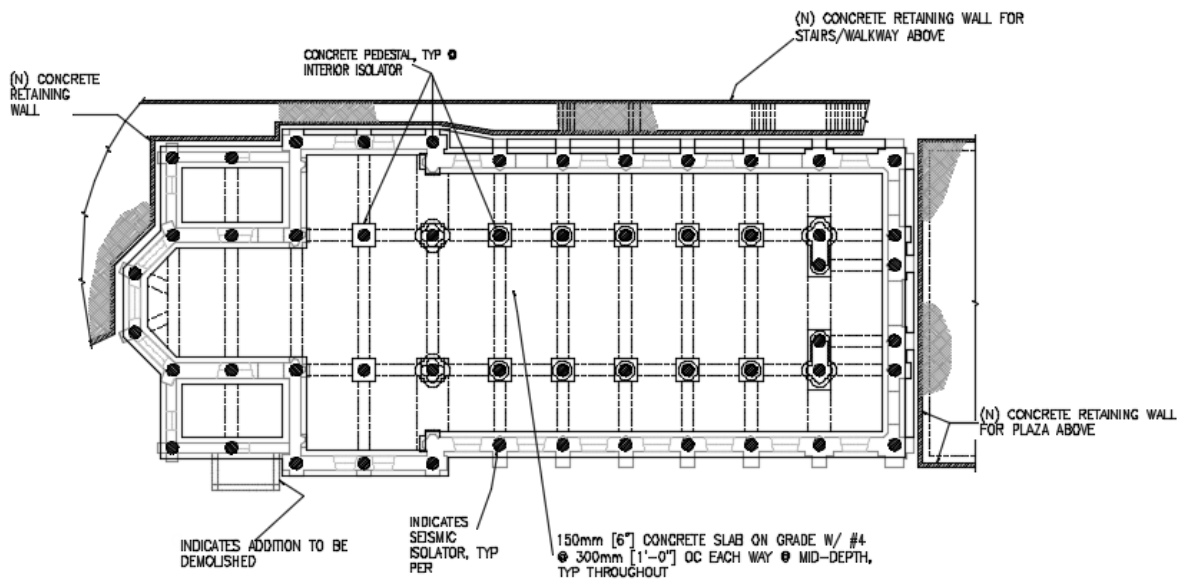


Figure A.35. Seismic retrofit by addition of seismic isolation

A.5 Seismic retrofitting for diaphragms and floors

Figure A.36 through Figure A.40 present example details for seismic retrofit of horizontal (diaphragm and collector) element of the lateral system

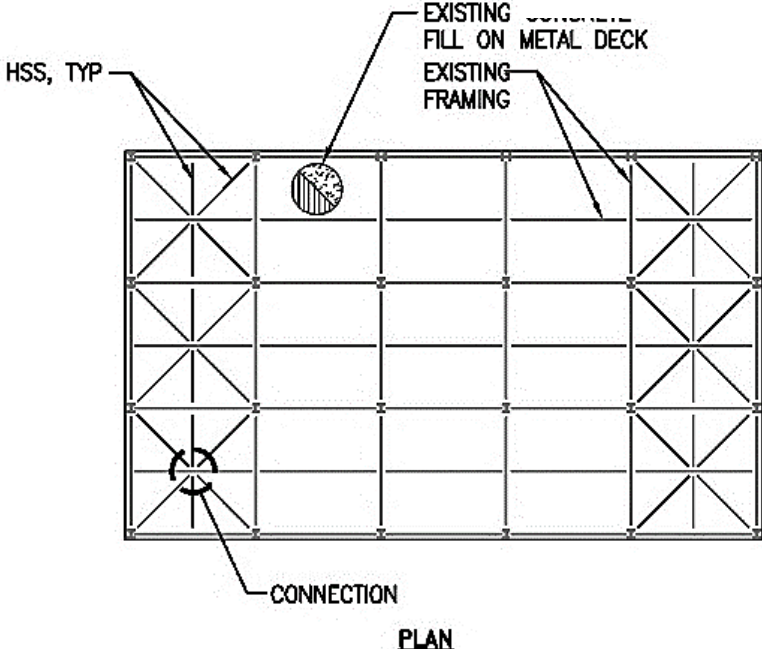


Figure A.36. Diaphragm bracing

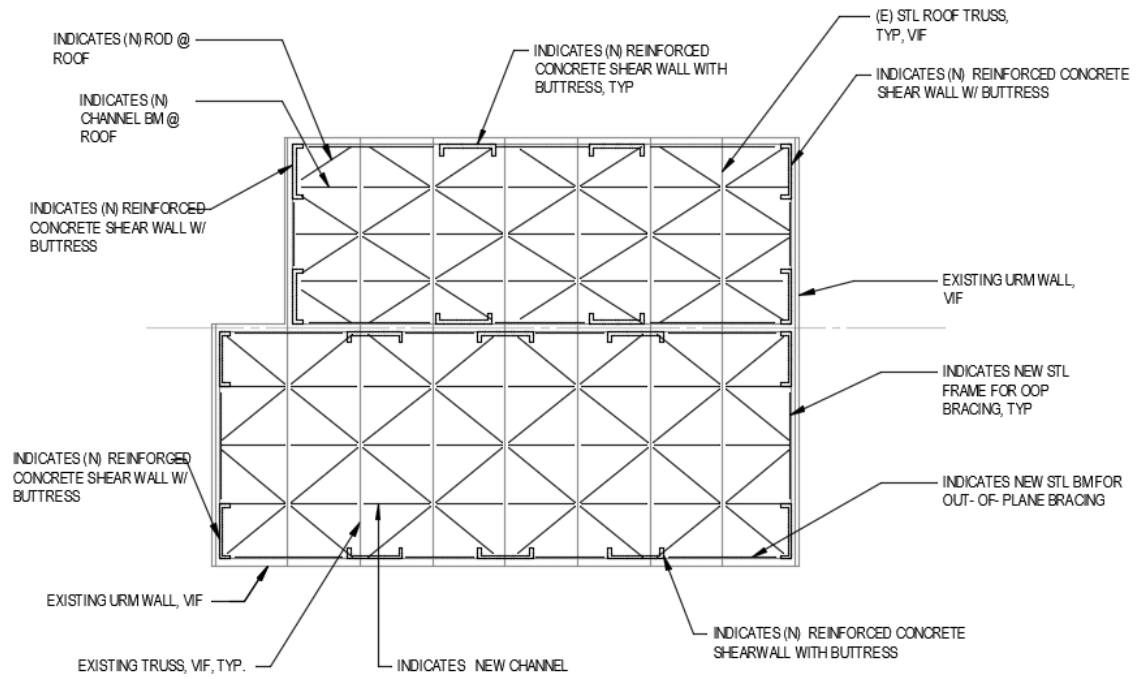
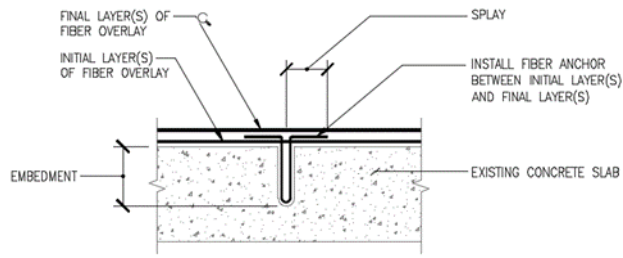
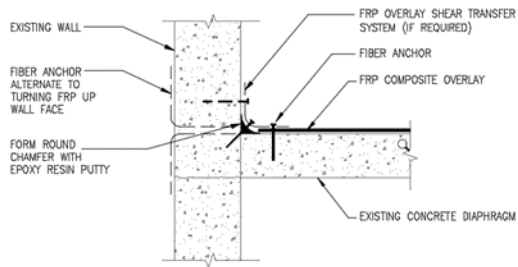


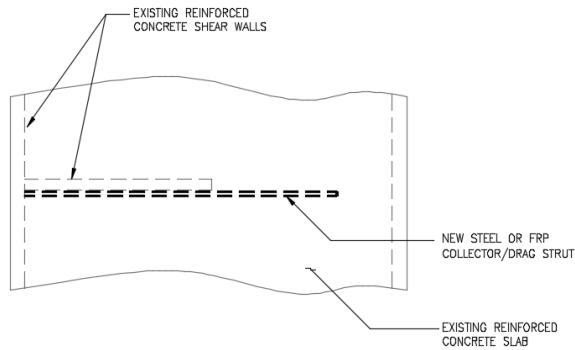
Figure A.37. Strengthening of Diaphragms roof level 1



FIBER ANCHOR DETAIL

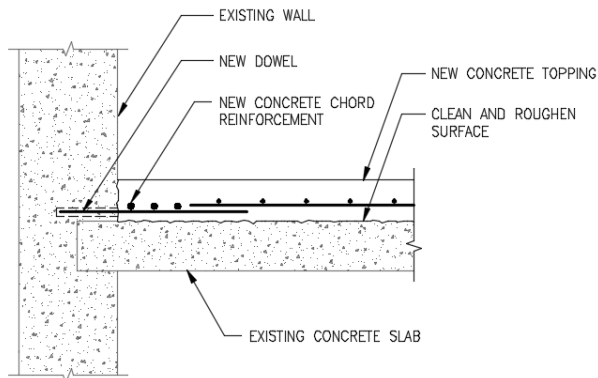


DIAPHRAGM FRP RETROFIT



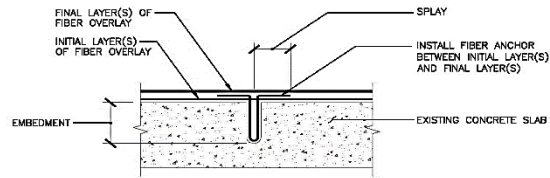
PARTIAL PLAN

STEEL ANGLE COLLECTOR

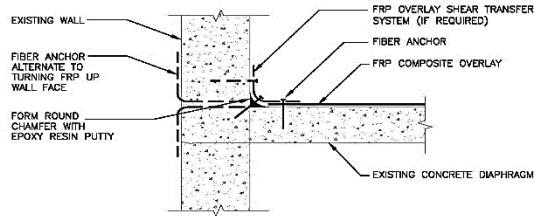


TOPPING SLAB

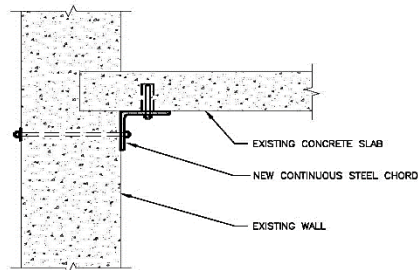
Figure A.38. Fiber Anchor and Diaphragm Strengthening



FIBER ANCHOR DETAIL

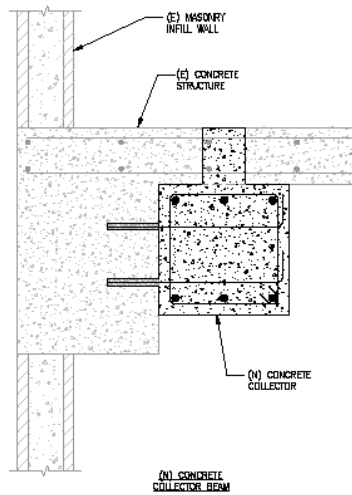


DIAPHRAGM FRP RETROFIT



STRENGTHENING SLAB TO WALL CONNECTION

Figure A.39. Diaphragm FRP retrofit



SECTION DETAIL

Figure A.40. Add new ductile concrete collector beam to existing concrete frame

A.6 Seismic retrofitting of foundations

Figure A.41 presents example details for seismic retrofitting of the foundations

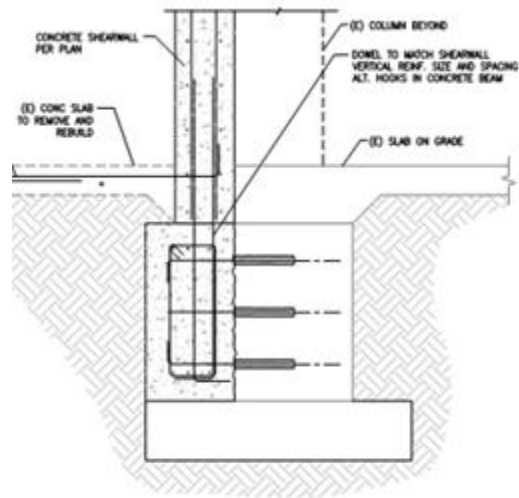
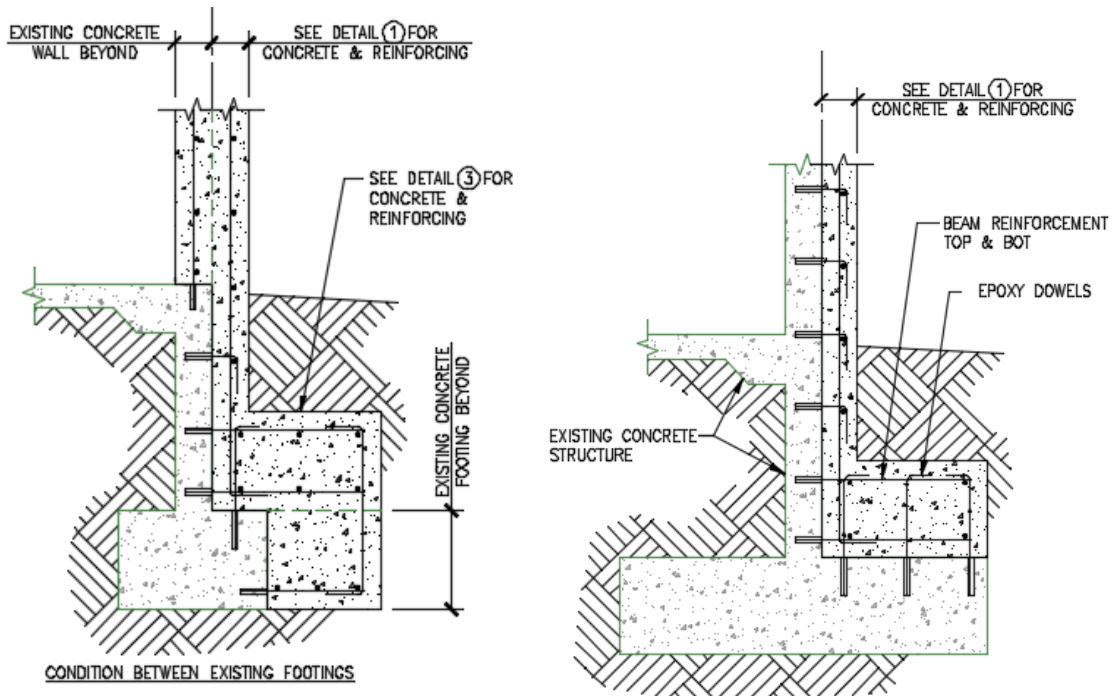


Figure A.41. Enlarge connection of grade beam with existing footing



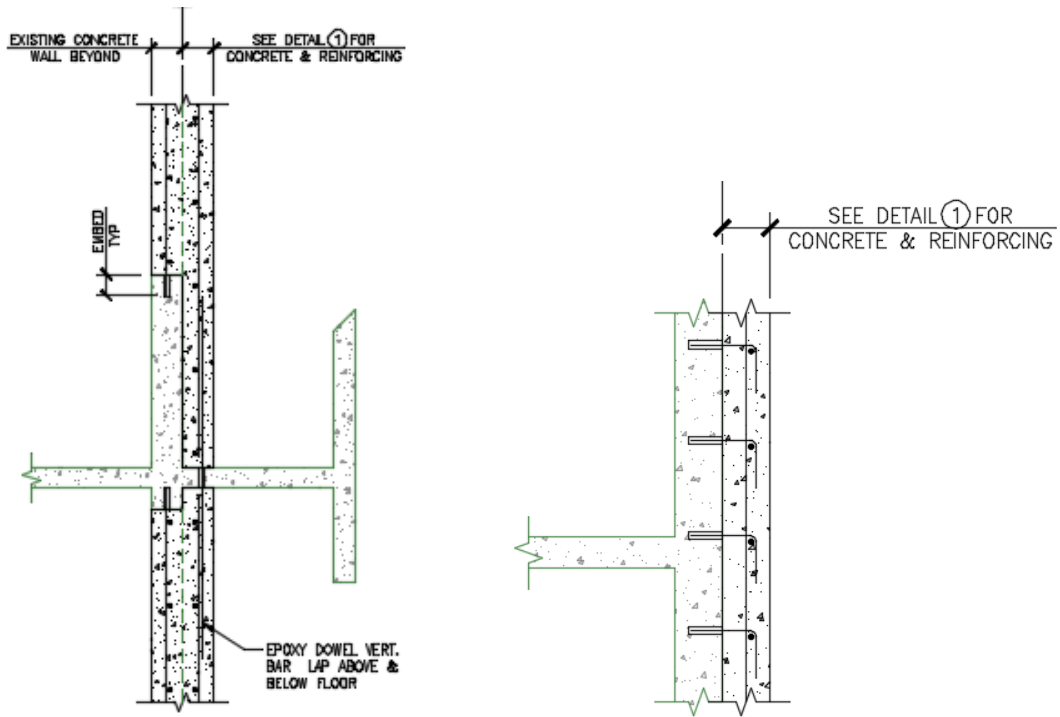


Figure A.42. Foundation and connection details for new walls

A.7 Seismic retrofit of nonstructural components

Figure A.43 through Figure A.50 presents examples of seismic-code compliant detailing for nonstructural components.

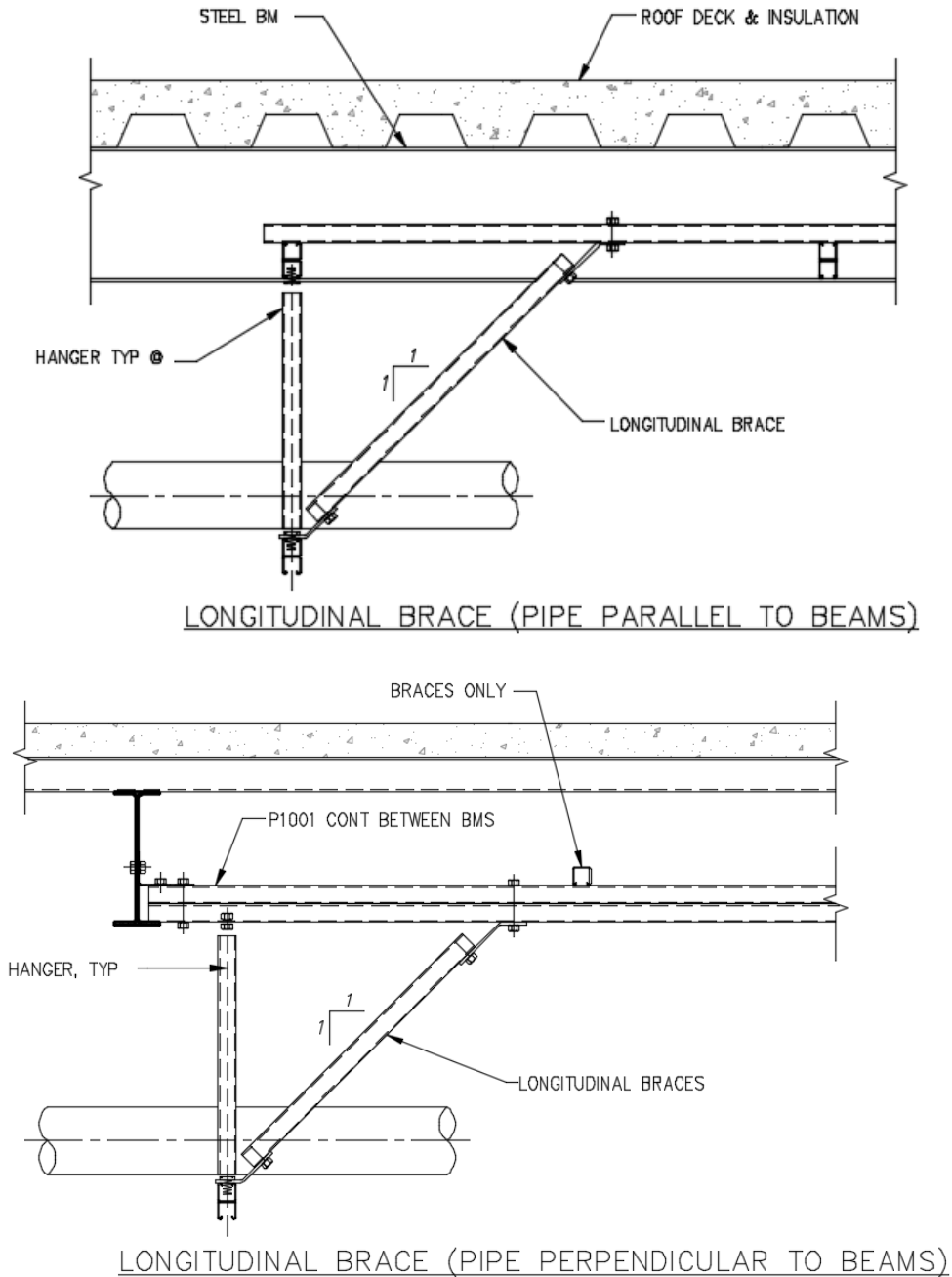


Figure A.43. Bracing and anchorage for pipes and ducts

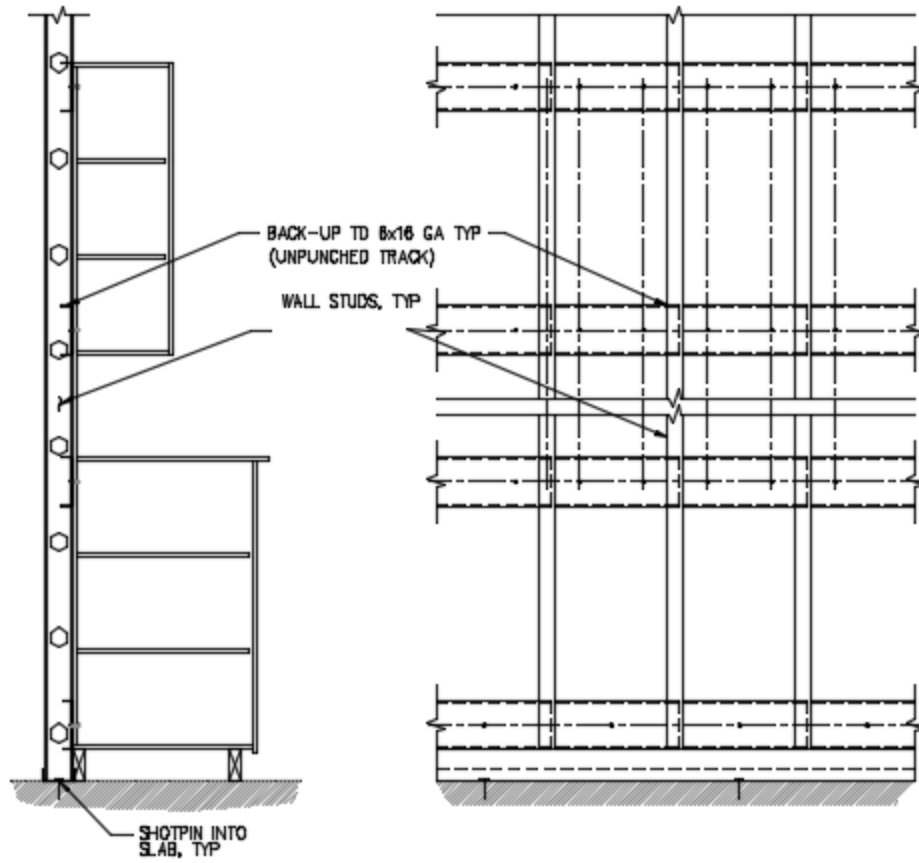
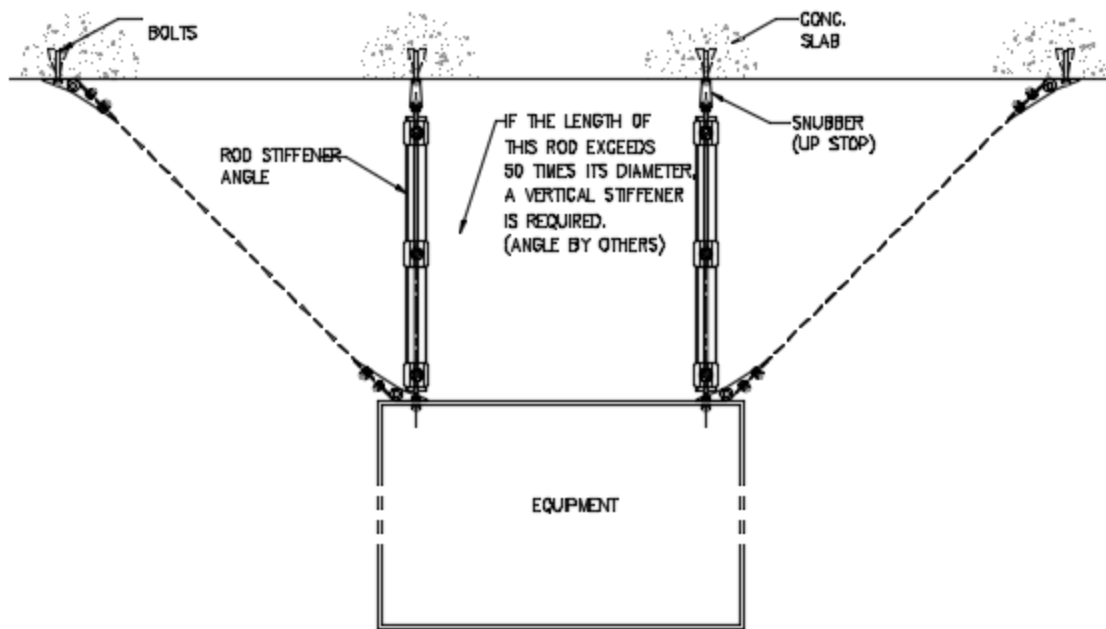
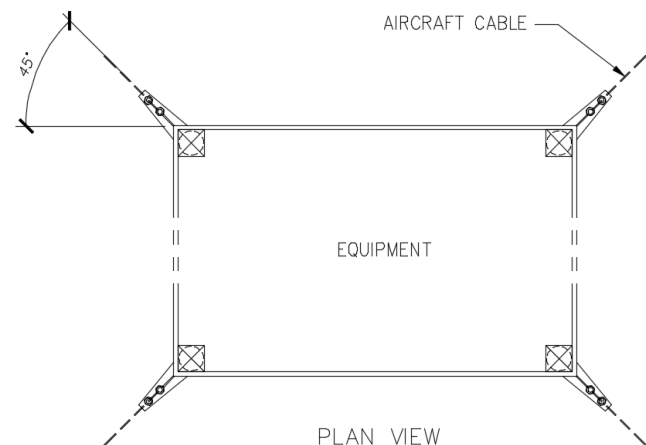


Figure A.44. File cabinet anchorage

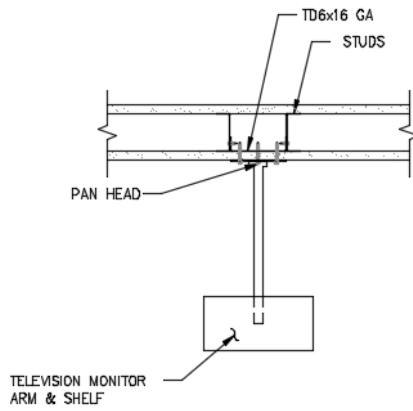


ELEVATION



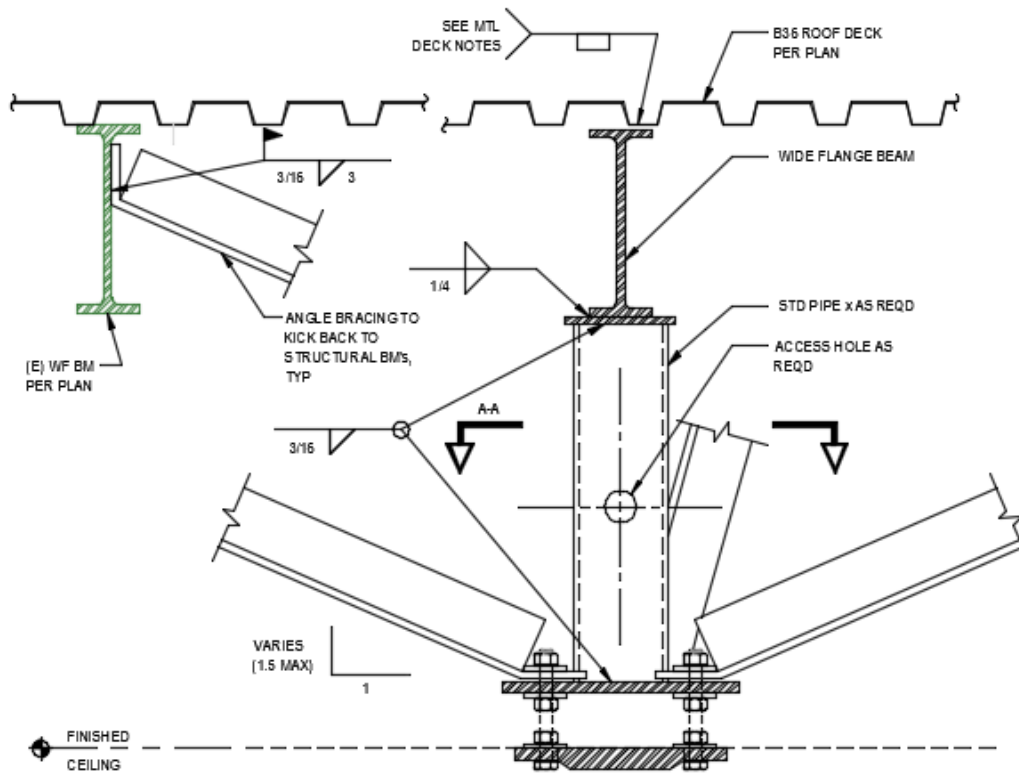
PLAN VIEW

Figure A.45. Suspended Equipment Support



PLAN

Figure A.46. Attachment of elevated monitor to the structure



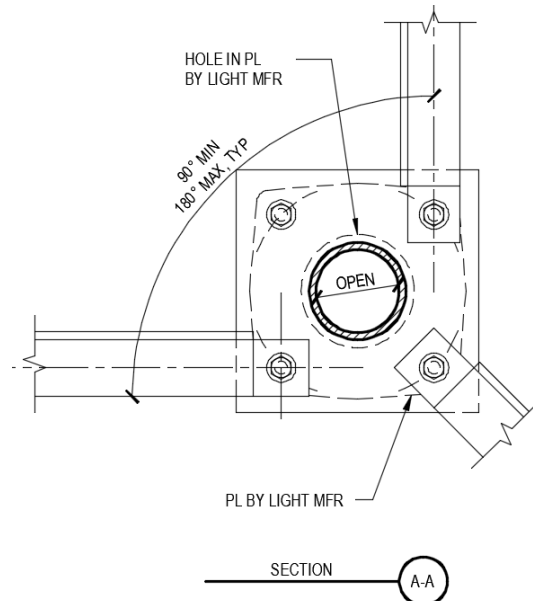


Figure A.47. Lighting support-Overhead light

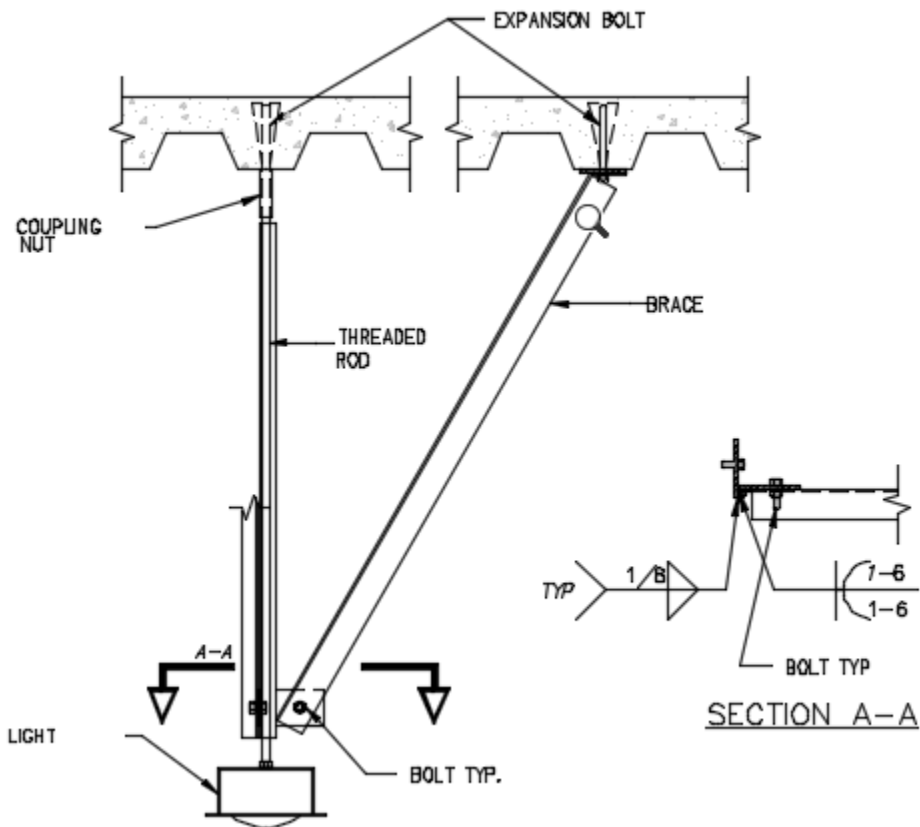


Figure A.48. Vertical and Lateral Lighting Support

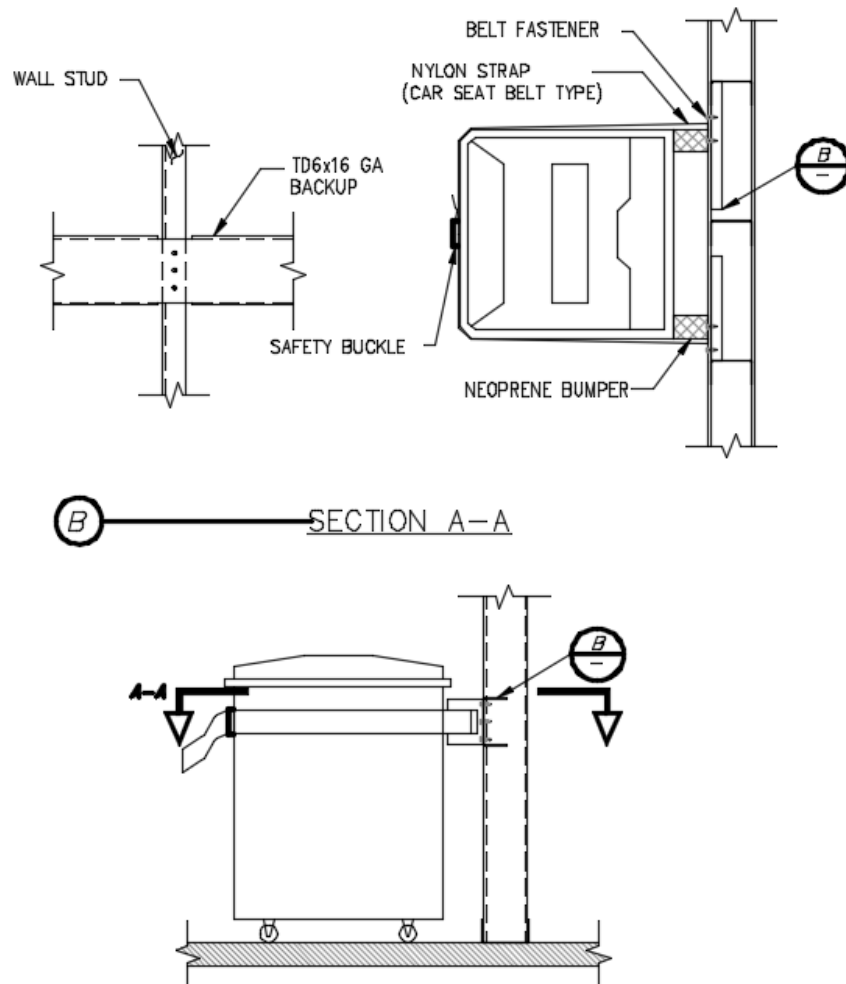


Figure A.49. Lateral Restraint of Equipment not in operation

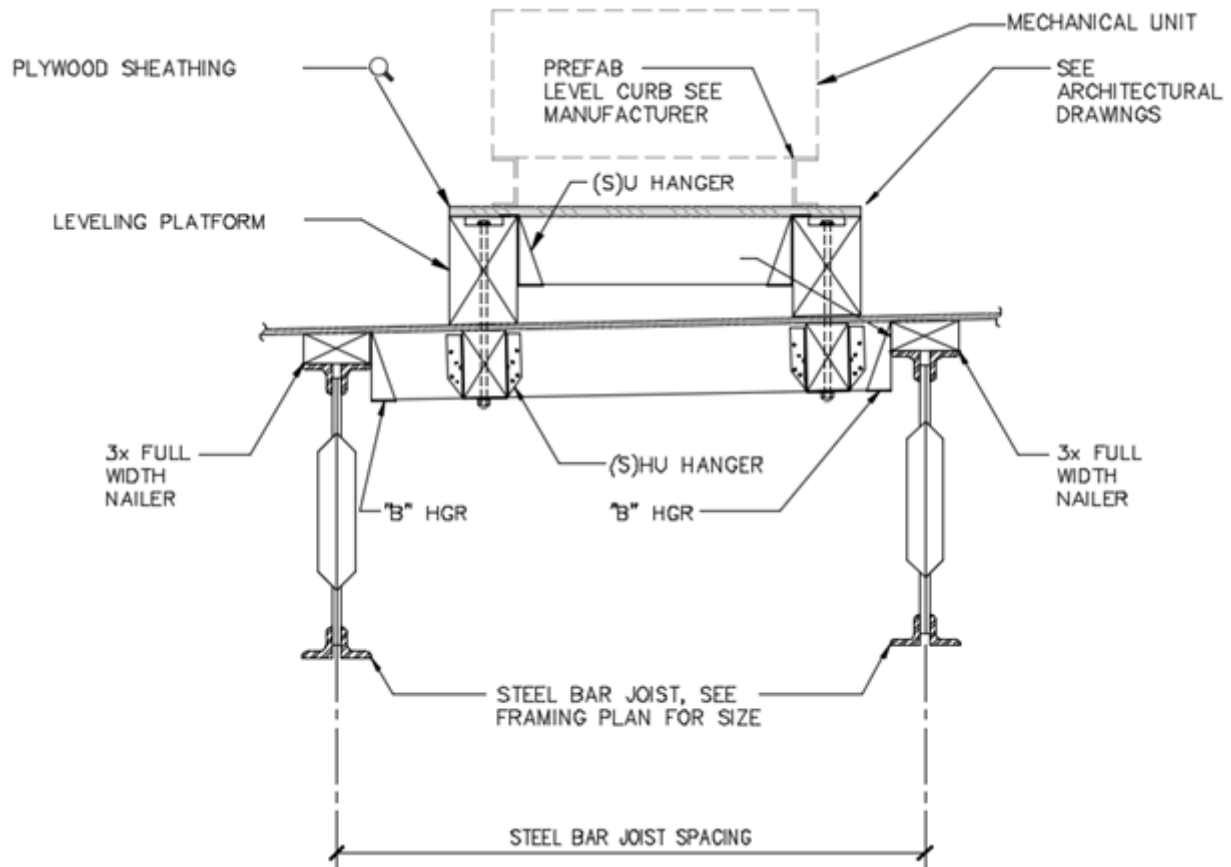


Figure A.50. Mechanical Unit supported from Ceiling

APPENDIX B SUMMARY OF REFERENCE DOCUMENTS

B.1 Current provisions of Nepal Code

B.1.1 Overview

After the destructive earthquake of M6.8 that struck eastern Nepal in 1988, the need for a national building code was first realized. Consequently, the Nepal National Building Code (NNBC) was developed by the Department of Urban Development and Building Construction (DUDBC) of the Ministry of Physical Planning and Works (MPPW) with the assistance of United Nations Development Program (UNDP) and United Nations Human Settlement Program (UN-HABITAT) and put into effect in 1994. The code was made mandatory in 2003 as a legally binding document in many municipalities.

In current NNBC, there are four levels of designs:

- International state-of-the-art –with the goal of allowing engineers in Nepal to use the most sophisticated level of design, the present code should not bar anyone who can produce high level of engineering;
- Professionally engineered building: These are the standard code requirements that all professionally qualified engineers will recognize and follow when designing structures in Nepal. It covers all major structures such as hospitals, meeting halls, factories, multi-story buildings and larger residential building, etc.;
- Rules of Thumb: This section recognizes that it is not practical at present to insist that professionals design all small buildings, and pre-engineered design plan can be used with rules of thumbs without sophisticated calculations; and,
- Advisory guidelines: Non-engineered constructions employing traditional methods and materials.

The NBC 105, 1994 is the code for the seismic design of buildings in Nepal that is used on professionally engineered buildings. Other codes listed within NNBC are mostly for mandatory Rules of Thumb or advisory guidelines.

All these codes are not complete and heavily rely on the relevant Indian Standards for their completeness. Most of the engineers use Indian Standards, considering that the whole of Nepal is Seismic Zone V as per the Indian Standards and including seismic loading standards for the design of buildings in Nepal.

B.1.2 NBC:105 Seismic design of building in Nepal

The NBC 105 is the main seismic design code of Nepal that sets down requirements for the general structural design and seismic loading for buildings for earthquake-resistant construction. The requirements of this section of the Nepal Building Code should be adopted in conjunction with IS 4326 - 1976 Code of Practice for Earthquake Resistant Design and Construction of Buildings.

Nepal's present NNBC 105 describes two methods for calculation of seismic forces: The Seismic Coefficient Method (static) and the Modal Response Spectrum Method. The resistant buildings are designed using equivalent static lateral forces to represent the effects of ground motion due to earthquake on buildings. The application of this method is limited to reasonably regular structures. The present code restricts the use of this method for structures up to 90 m height, and should also mention the condition of regularity. The Modal Response Spectrum Method is basically used for normal structures over 40 meters high and with irregular configuration. The dynamic analysis is confined to the response spectrum method. The Time History Analyses (linear and nonlinear) is not covered in Nepal codes.

B.1.3 NBC:201 Mandatory rules of thumbs reinforced concrete building with masonry infills

The main NBC201, Mandatory Rules of Thumb (MRT), is a design code to provide ready-to-use dimensions and details for various structural and non-structural elements for building up to three-story

reinforced concrete (RC), framed, ordinary residential buildings commonly being built in Nepal using brick infill walls. This MRT is intended for use by the mid-level technicians who are not trained to undertake independently the structural design of buildings; and also to civil engineers who want to use this document for effective utilization of their time by using the design procedures outlined here. Compliance with the MRT leads to the present non-engineered construction being superseded by pre-engineered designs, which should achieve acceptable minimum seismic safety requirements (such as those specified by NBC 105 and IS 1893-1884 etc.).

B.1.4 NBC 202: Mandatory Rules of Thumb for load bearing masonry

The NBC202 Mandatory Rules of Thumb for load-bearing masonry building are used in the design non-engineered buildings in Nepal as following:

- One or two stories, if built of fired brick in mud mortar, or stone masonry in cement or mud mortar
- Three stories, if built of fired brick in a cement mortar.

This document provides suitable illustrations to explain the important points, sketches and sufficient data to proportion the critical strength elements correctly. The requirements are based on pre-engineered design calculations of typical structures meeting prescribed criteria.

B.1.5 NBC: 203 Guidelines for earthquake resistant building construction low strength masonry

This document provides basic guidelines for the earthquake resistance of low-strength masonry (LSM) construction. This is used for all types of LSM public buildings to be built throughout Nepal. This code is widely used for all LSM residential buildings to be built in Municipal and urban areas where the building permit process exists.

B.1.6 NBC: 204 Guidelines-earthquake resistant buildings construction earthen buildings

This document provides basic guidelines for the earthquake resistance of earthen buildings. The recommendations set forth in this standard are Mandatory Rule of Thumb for all types of public earthen buildings to be built throughout Nepal.

B.1.7 NBC: 205 Mandatory rules of thumb for reinforced concrete buildings without masonry infill

The main objective of these Mandatory Rules of Thumb (MRT) is to provide ready-to-use dimensions and details for various structural and non-structural elements for up to three-story reinforced concrete (RC), framed, ordinary residential buildings commonly being built by owner-builders in Nepal. Their purpose is to replace the non-engineered construction presently adopted with pre-engineered construction so as to achieve the minimum seismic safety requirements specified by NBC 105. This MRT is intended for mid-level technicians who are not trained to undertake independently the structural design of buildings

B.2 Indian Standards

B.2.1 Overview

India has the following seismic design codes: IS 1893 (Part 1): 2002 ‘Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings’, IS 4326: 1993 ‘Code of practice for earthquake resistant design and construction of buildings’ and IS 13920: 1993 ‘Ductile detailing of reinforced concrete structures subjected to seismic forces — Code of practice’

B.2.2 IS:1893: 2002 Criteria for earthquake resistant design of structure

The first Indian seismic code, IS 1893 “Recommendation for earthquake resistant design of structures,” was published in 1962 and has been revised in 1966, 1970, 1975, 1984, 2002 and most recently revised in 2016. The code has been split into a number of parts, with the first part containing general provisions; those pertaining to buildings was released in 2002. This part is for the general provisions (applicable to all structures) and specific provisions for buildings.

The code IS1893:2002 (recently revised into 2016) is the main earthquake resistant design code in India for buildings and other structures.

The recently released IS 1893:2016 is a more comprehensive form. In this revised version of IS 1893 (Part 1): 2016, the following significant changes have been included:

- Design spectra are defined for natural period up to 6s;
- Same design response spectra are specified for all buildings;
- Bases of various load combinations have been made consistent with those specified in the other codes;
- Temporary structures are brought under the purview of this standard;
- Importance Factor provisions have been modified;
- A provision is introduced to ensure that all buildings are designed for at least a minimum lateral force;
- Buildings with flat slabs are brought under the purview of this standard;
- Additional clarity is given on how to handle different types of irregularities of structural systems;
- The effect of masonry infill walls has been included in design of frame buildings;
- A method is introduced for arriving at the approximate natural period of buildings with basements, step back buildings and buildings on hill slopes;
- Torsional provisions are simplified;
- Simplified method is introduced for liquefaction potential analysis.

B.2.3 IS15988: 2013- Seismic evaluation and strengthening of existing reinforced buildings

The IS 15988: 2013 - Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings - Guidelines is the current standards in India. The seismic performance of existing buildings is evaluated in relation to the performance criteria in use for new buildings. The provisions of this standard are strongly correlated with the design criteria of new buildings contained in IS 1893 (Part 1). There are two levels of evaluation: Preliminary evaluation and detailed evaluation. The preliminary evaluation is a quick procedure to identify the potential earthquake risk of a building and to screen buildings for detailed evaluation. In this evaluation, there are configuration-related checks and strength-related checks. The detailed evaluation procedure is based on determining the probable strength of lateral load resisting elements and comparing them with the expected seismic demands. The detailed evaluation is compulsory for buildings more than 6 stories; buildings located on incompetent or liquefiable soils and/or located near (less than 15 km) active faults and/or with inadequate foundation details; and buildings with inadequate connections between structural members.

B.2.4 IS: 1905-1987-Code of practice for structural use of unreinforced masonry

This Indian Standard (Third Revision) was adopted by the Bureau of Indian Standards on 30 August 1987, after the draft finalized by the Structural Safety Sectional Committee had been approved by the Civil Engineering Division Council. This standard gives recommendations for the structural design aspect of unreinforced load-bearing and non-load bearing walls, constructed with solid or perforated burnt clay bricks, sand-lime bricks, stones, concrete blocks, lime based blocks or burnt clay hollow blocks in regard to the materials to be used, maximum permissible stresses and the methods of design

Structural adequacy of masonry walls depends upon a number of factors, among which mention may be made of quality and strength of masonry units and mortars, workmanship, methods of bonding, unsupported height of walls, eccentricity in the loading, position and size of openings in walls as well as location of cross walls and the combination of various external loads to which walls are subjected. The recommendations of the code do not apply to walls constructed with mud mortars.

B.3 Retrofit guidelines

B.3.1 General

After Nepal Gorkha Earthquake in 2015, DUDBC/MoUD has produced – “Seismic Retrofitting Guideline of Building in Nepal.” This guideline was prepared with the objective of strengthening existing housing stock. The purpose of this document is to provide an analysis and design methodology for use in the seismic evaluation and retrofit of the existing buildings in Nepal.

This manual is being prepared in three separate volumes to provide retrofitting guidelines for adobe structures, masonry structures and RCC structures covering both theoretical and practical aspects of retrofitting. It basically focuses on seismic retrofitting and strengthening techniques. The document references the Seismic Vulnerability Evaluation Guideline for Private and Public Buildings which has been adopted by DUDBC.

B.3.2 Seismic retrofit guideline of Nepal, 2016, Vol (1)-Adobe and low strength masonry

The purpose of this document is to provide an analysis and design methodology used in the seismic evaluation and retrofit of existing adobe and low-strength masonry buildings in Nepal. This guideline includes concepts of repair, restore and retrofitting of buildings, common damages in adobe and low-strength masonry structures, and retrofitting techniques on different elements with some hand calculation and construction techniques with sketches and photos. For the techniques, it includes both engineering as well as local technologies and materials such as bamboo, PP band and recycled tires etc.

B.3.3 Seismic retrofit guideline of Nepal, 2016 Vol (II) Masonry structure

This guideline focuses on load-bearing masonry structures, especially brick masonry buildings. It also includes common damages and failure patterns in masonry structures, retrofitting criteria, analysis process and methods, and retrofitting and strengthening techniques for different components of masonry buildings. It briefly discusses different analysis methods: Elastic analysis (both linear static and linear dynamic procedures), inelastic analysis (non-linear static) and non-linear analysis, as well as performance base behaviour of masonry structures. It includes the hand calculation of buildings to check stress and design retrofitting measures.

B.3.4 Seismic retrofit guideline of Nepal, 2016 Vol (III) RCC Structure

This guideline basically focuses on structural evaluation and retrofitting design moment frame RCC structures. For the structural evaluation, it briefly discusses three-tier evaluations based on FEAM – Rapid Visual Inspection/ assessment, preliminary assessment, and details assessment. The detailed evaluation procedure is based on determining the probable strength of lateral load resisting elements and comparing them with the expected seismic demands. It also briefly describes the required three performance levels of structural and non-structural components. It further illustrates seismic retrofitting strategies for improved performance in future earthquakes.