SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL

2016

RCC

Government of Nepal
Ministry of Urban Development
Department of Urban Development and Building Construction
Babarmahal, Kathmandu

MINISTRY OF URBAN DEVELOPMENT, 2016

The Seismic Retrofitting Guidelines of Buildings in Nepal has been developed by Center of Resilient Development (CoRD) and MRB Associates with support from UNDP/Comprehensive Disaster Risk Management Programme.

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SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL, 2016
RCC STRUCTURES

Government of Nepal

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MESSAGE

I am glad to know that the Ministry of Urban Development is publishing the "SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL (ADOBE)," SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL (MASONRY)" and " SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL (RCC)". The aim of these documents is to guide and facilitate the retrofitting works of buildings to make them earthquake resistant and thereby reducing the risk of life and injury during an earthquake.

Recent Gorkha Earthquake 2015 claimed 8,790 lives in total and more than 22,300 were injured. More than 500,000 residential buildings and 2656 official buildings were collapsed completely and almost 200,000 residential buildings and 3,622 official buildings were partially damaged. Likewise more than 19,000 rooms of school building were completely damaged and more than 11,000 rooms were partially damaged. More than 2,900 numbers of temples and social buildings have been severely affected including all seven world heritage sites in Kathmandu.

This retrofitting guideline will be a useful document for those existing building which are partially damaged and can be re-used through retrofitting and can also be used in controlling the extent of damage of an existing structure.

I would like to encourage the practitioner, technical persons, designers and Engineers to follow the guidelines who are involved in retrofitting and construction works of buildings and would like to request for the media persons too for highlighting the usefulness of this document for safer building construction in our nation.

I would like to acknowledge the efforts made by the staffs of Ministry of Urban Development, and Department of Urban Development and Building Construction who have given their valuable feedback and guidance. Likewise I would also like to extend my gratitude to United Nations Development Program (UNDP) and Center of Resilience Development (CoRD) and MRB Associates for their support to prepare these documents.

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FOREWORD

Nepal is located between two active tectonic plates, Indian and Tibetan, where the Indian plate is sub-ducting at a rate of about 3 cm per year below the Tibetan plate. The existence of the young Himalayan range is an evidence of this continued uplift. As a result of this tectonic movement, Nepal lies in the most seismically active zone.

The past history of frequency and intensity of earthquake have exposed the vulnerability and coping capacity of the nation. The damage incurred during the earthquakes has been massive, for instance in 1988, the 6.7 magnitude earthquake killed 721 people and 7000 buildings were destroyed. Recent Gorkha Earthquake 2015 claimed 8,790 lives in total and more than 22,300 were injured. More than 500,000 residential buildings and 2,656 official buildings were collapsed completely and almost 200,000 residential buildings and 3,622 official buildings were partially damaged. These earthquakes highlighted a need for preparation of the National Building Code to ensure structural safety of the buildings, though it was formally enforced only in 2004.

While the implementation of building code has been a challenge, there is already a significant stock of non-engineered, semi–engineered structures, built before the code was implemented that need to be strengthened for withstanding the future earthquake. This document - "Retrofitting Guideline" has been developed to fill this gap.

The objective of this document is to reduce risk to life and injury during an earthquake damage or to control the extent of damage of existing structures. This will be a guiding document for the design professionals with the primary purpose of providing analysis and design methodology for use in the seismic evaluation and retrofitting of the existing buildings in Nepal. This manual is being prepared in three separate volumes providing retrofitting guidelines for adobe structure, masonry structure and RCC structure covering both theoretical and practical aspects of retrofitting.

I would like to acknowledge the efforts made by Mr. Shiva Hari Sharma, Joint Secretary and Mr. Pramod Krishna Karmacharya, undersecretary of Ministry of Urban Development. Likewise the staffs of Department of Urban Development and Building Construction who have given their valuable feedback and guidance also deserve recognition. I would also like to extend my gratitude to Mr. Vijaya Singh, Assistant Country Director of United Nations Development Program (UNDP) and Center of Resilience Development (CoRD) and MRB Associates for their support to prepare these documents. Last, but not the least, I would also like to extend my gratitude to all the professionals, who were engaged in the process of preparation of this document for giving it a final shape.

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ACKNOWLEDGEMENT

It gives me immense pleasure for the publication of Seismic Retrofitting Design Guidelines of Buildings in Nepal. This Guideline is the first attempt for the Government of Nepal to guide the respective practitioner and academician for making the structure safer.

I expect that this guideline will be useful for the Designers as well as Engineers in general who are involved in retrofitting design and construction of buildings. This guideline will also helpful in raising the safety awareness and making to community disaster resilient.

My sincere thanks goes to the respected Secretary Mr. Deependra Nath Sharma, Joint Secretary Mr. Shiva Hari Sharma, Senior Divisional Engineer Mr. Pramod Krishna Karmacharya and all the personnel involved directly or indirectly for preparing of this design guideline.

Also, my thanks go to United Nations Comprehensive Disaster Risk Management Program, CDRMP for the support during the preparation of the guideline and publication as well.

At last, but not the least, I would like to thank Center of Resilience Development (CoRD) and MRB Associates for their support and preparing this design guideline.

RPSingh

(Dr. Ramesh Prasad Singh)
Director General

FOREWORD

Nepal is home to the breathtaking Himalayas which as the world's youngest ranges are growing a few centimeters each year due to the uplift caused by the northward push of the Indian tectonic plate against the Eurasian plate. This manifests in large magnitude earthquakes recurring at a periodicity of 7 to 8 decades, and situates the entire country in a high seismic risk zone and its ranking as 11th in terms of its relative vulnerability to earthquake.

UNDP's Comprehensive Disaster Management Programme (CDRMP), taking cognizance of these risks and those posed by climate variability and change was formulated in 2011. It aims to strengthen the institutional and legislative sectors of Disaster Risk Management in Nepal by building the capacity of the key government ministries, its line agencies and local bodies. It also aims to enhance resilience in Nepal by strengthening partnership with national, institutional and the private sector, civil society and other development actors for Disaster Risk Management including Climate Change Adaptation.

Seismic Risk Reduction continues to be a key area of UNDP's collaboration with Government of Nepal from over two decades ago. UNDP contributed to preparation of National Building Code, the development of curricula and manuals for training of engineers and masons to implement provisions of the building code and implementation of code-compliant building permit systems in several municipalities in Nepal. Kathmandu Metropolitan City has recently fully operationalized the electronic Building Permit System with technical assistance from UNDP and funding support from UK Aid. Through Nepal Risk Reduction Consortium, UNDP co-led the formulation of a National Action Plan for Safer Building Construction.

Learning from our engagement on retrofitting of schools in Illam and Taplejung districts post-2011 Sikkim earthquake, and realizing the need for strengthening existing vulnerable buildings, UNDP in collaboration with Department of Urban Development and Building Construction (DUDBC) of Ministry of Urban Development (MOUD) formulated the Retrofitting Guideline. This guideline was prepared to strengthen existing housing stock to cope with seismic shocks with technical support from Center of Resilient Development (CORD) and Manohar Rajbhandari Associates, and with active engagement of officials of DUDBC. The guideline encompasses three volumes addressing the three dominant construction typologies namely Adobe and low strength masonry, Masonry and RCC construction.

The April 2015 Gorkha Earthquake exposed the significant vulnerability of the existing buildings in both urban and rural areas and highlighted importance for such a guideline resulting in MOUD approving the guidelines in October this year. We hope this will help to undertake trainings and necessary repair, restoration and retrofitting of buildings damaged by April 2015 earthquake. It could also serve as a guiding document for practitioners, engineers and designers to undertake retrofitting measures to reduce risk to future disaster. This guideline is an initial yet important step as we embark on this journey to create safe and resilient buildings and settlements in Nepal.

I would like to extend my sincere acknowledgement to Mr. Shiva Hari Sharma, Joint Secretary, MOUD and Mr. Ramesh Prasad Singh, Director General DUDBC and their team for their valuable feedback and guidance throughout the process of formulation and approval of the guidelines. I would also like to thank Dr. Hari Darshan Shrestha, Dr. Jishnu Subedi and Mr. Manohar Rajbhandari for their technical support in preparation of this guideline, and acknowledge contributions of my colleagues at UNDP in this endeavor.

Valerie Julliand
UNDP Resident Representative &
United Nations Resident Coordinator

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LIST OF ACRONYMS

Sa/g	Average response acceleration coefficient
Ac	Summation of cross-sectional area of all columns in the
Ac	Actual concrete to be provided in the jacket
Ah	Design horizontal seismic coefficient
As	Actual steel to be provided in the jacket
C	Basic seismic coefficient
CFRP	Carbon Fiber Reinforced Polymer
CoRD	Center of Resilient Development
CP	Collapse Prevention
d	Base dimension of the building at plinth level
DCR	Demand Capacity Ratio
dh	Diameter of stirrup
DL	Dead load
EDU	Energy Dissipation unit
F0	Axial stress of column due to overturning forces
fck	Characteristic strength of concrete
FRP	Fiber Reinforced Polymer
fy	Yield strength of steel
H	Height of the building
hi	Height of floor 'i' measured from base
I	Importance Factor
IO	Immediate Occupancy
IS	Indian Standard
K	Structural performance factor
L	Length of the building
LL	Live load
LS	Life Safety
M	Moment of resistance
NBC	National Building Code
nc	Total number of columns resisting lateral forces in the
nf	Total number of frames in the direction of loading
P	Axial load
p	Percentage of steel
R	Response reduction factor
RC	Reinforced Concrete
RCC	Reinforced Cement Concrete
S	Spacing of ties to be provided in the jacket
Ta	Natural time period of vibration
Tcol	Average shearing stress in column
tj	Thickness of Jacket
V or VB	Design seismic base shear
Vj	Maximum storey shear at storey level 'j'
W or Wt	Seismic weight of the building
W¬i	Proportion of Wt contributed by level 'i'
Z	Zone Factor
UNDP	United Nations Development Programme

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1. INTRODUCTION

1.1. BACKGROUND

Nepal is located in the boundary between the Indian and Tibetan plates, along which a relative shear of about 2 cm per year has been estimated. The Indian plate is also sub-ducting at a rate of, thought to be, about 3 cm per year. The existence of the Himalayan range with the world's highest peak is an evidence of continued uplift. As a result, Nepal is seismically very active. Nepal lies in the seismic zone V which is the most vulnerable zone.

As Nepal lies in the seismic prone area and earthquake occurs frequently, people here in Nepal are now more earthquake concern. The damages caused by earthquake, small damage or large damage show the vulnerability of buildings in Nepal.

The structures of Nepal are mostly non-engineered and semi – engineered construction, which are basically lack of seismic resistance detailing. The main causes of above are lack of awareness of seismic resistance importance and strictly implementation of the codes by government level.

The non -engineered, semi –engineered structures or structures which were built before the building code was implemented can be rebuilt or reconstructed to reduce certain degree of seismic vulnerability.

1.2. PURPOSE

The primary 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. It is expected that this document will be used by retrofit design professionals performing seismic evaluations and retrofit designs.

1.3. OBJECTIVE AND SCOPE

The objective of this document is to reduce risk of loss of life and injury. This is accomplished by limiting the likelihood of damage and controlling the extent of damage.

Buildings are designed to perform at required performance level throughout its life. The material degradation due to aging and alterations carried out during use over time necessitates the operations like Repair, Restoration and Retrofit. The decay of building occurs due to original structural inadequacies, weather, load effects, earthquake, etc.

2. CONCEPT OF REPAIR, RESTORATION AND RETROFITTING

2.1. REPAIR

Repair is the process to rectify the observed defects and bring the building to reasonable architectural shape so that all services start to function. It consists of actions taken for patching up superficial defects, re-plastering walls, repairing doors and windows and services such as following:

i. Patching up of defects as cracks and fall of plaster and re-plastering if needed.
ii. Repairing doors, windows, broken glass panes, etc.
iii. Rebuilding non-structural walls, chimneys, boundary walls
iv. Relaying cracked flooring at ground level, tiles
v. Redecoration work
vi. Re-fixing roof tiles

It would be seen that the repairing work carried out as above does not add any strength to the structure. In fact, repair will hide the existing structural defects and hence do not guarantee for good performance when the structure is shaken by an earthquake.

2.2. RESTORATION

Restoration aims to restore the lost strength of structural elements of the building. Intervention is undertaken for a damaged building by making the columns, piers, beams and walls at least as strong as original.

Some of the common restoration techniques are:

i. Removal of portions of cracked masonry wall and piers, and rebuilding them in richer mortar. Use of non-shrinking mortar will be preferable.
ii. Adding wire mesh on either side of a cracked component, crack stitching etc. with a view to strengthen it.
iii. Injecting neat slurry or epoxy like material, which is strong in tension, into the cracks in walls, columns, beams, etc.

2.3. SEISMIC STRENGTHENING (RETROFITTING)

When the existing building is incapable of withstanding the earthquake forces, it requires to be re-strengthened for safety. The complete replacement of such buildings in a given area may not be possible due to the historical importance or due to financial problems. Therefore, seismic strengthening of existing undamaged or damaged buildings is a definite requirement. The strengthening works must be fully justified from the cost point of view.

Retrofitting is undertaken to enhance the original strength to the current requirement so that the desired protection of lives can be guaranteed as per the current codes of practice against possible future earthquakes. Retrofitting of a building will involve either component strength enhancement or structural system modification or both. It is expected to improve the overall strength of the building.

2.3.1. MATERIAL AND CONSTRUCTION TECHNIQUES

Material and construction techniques are often done after damaging earthquake for repair and strengthening of the structure. Even though cement and steel are most commonly used as repair and strengthening materials, some of the techniques and material might not be familiar to the designer.

Material and construction techniques are often done after damaging earthquake for repair and strengthening of the structure. Even though cement and steel are most commonly used as repair and strengthening materials, some of the techniques and material might not be familiar to the designer.

2.3.1.1. Conventional Cast-in-Situ Concrete

Conventional cast in situ concrete process is used in repair and strengthening works in the cases where due to the change in volume or shrinkage of the convection cement based concrete, causing unsatisfactory results. The change in volume results in loss of good contact between the new concrete and the old element preventing sound transfer of stress at the contact surface. In order to improve bond characteristics and minimize the shrinkage, it is recommended to use higher strength concrete with low slumps and minimum water. In cases where super plasticizer are used to reduce shrinkage, a slump of about 20 cm is expected, while without super plasticizers the slump should not exceed 10 cm, using standard Abrams cone.

Placement techniques are very important with cast in situ concrete to insure that the new concrete will perform adequately with the older materials. Existing surfaces which will be in contact with new cast in situ concrete must be thoroughly roughened and cleaned for good bonding characteristic(fig: 2.2). After anchorages are installed(fig:2.1), forms are constructed to meet the desired surfaces. Special chutes or access hole are frequently required in the forms to allow the placement of concrete. Immediately before placement, a final cleaning of the form is essential to remove all sawdust, etc. and the existing concrete should be moistened. The concrete should be thoroughly vibrated to insure that it completely fills the forms and voids or rock pockets are avoided. Proper curing of the newly cast concrete is also important to prevent rapid drying of the surface.

Photo Source: MRB & Associates

Figure 2.1 Anchors driven inside concrete after placing epoxy resin

Photo Source: MRB & Associates

Figure 2.2 Roughing of concrete surface for proper bondage with new concrete

2.3.1.2. Shotcrete

Shotcrete is the method of repair and strengthening reinforced concrete member where mortar is forcefully sprayed through nozzle on the surface of the concrete member at high velocity with the help of compressed air. With shotcrete method a very good bond between new shotcrete and old concrete can be obtained while repair and strengthening process. This method can be applied vertically, inclined, and over head surfaces with minimum or without formwork. Generally the materials used in this method are same as conventional mortar, and reinforcement are welded fabric and deformed bars tacked onto surface.

Shotcrete process is carried out either by these two processes:
a) Wet process
b) Dry process

a) Wet process:
In the wet process mixture of cement and aggregate premixed with water and the pump pushes the mixture through the hose and nozzle. Compressed air is introduced at nozzle to increase the velocity of application.

b) Dry process:
In dry mix process, compressed air propels premixed mortar and damp aggregate and at the nozzle end water is added through a separate hose. The dry mix and water through the second hose are projected on to a prepared surface.

Surface preparation before shotcreting involves a thorough cleaning and removing all loose aggregate and roughening the existing concreting surface for improved bond. Shotcrete frequently has high shrinkage characteristics and measures to prevent cracks using adequate reinforcement and proper curing is always necessary. The shotcrete surface can be lift as sprayed which is somewhat rough. If a smoother surface is required, a thin layer can be sprayed on the hardened shotcrete and then reworked and finished to the required texture or plaster can be applied.

The equipment required for a minimum shotcrete operation consists of the gun, an air compressor, material hose, air and water hose, nozzle, and some time a water pump. Miscellaneous small hand tools and wheelbarrows are also required. With this minimum equipment, shotcrete works can be accomplished satisfactorily.

2.3.1.3. Grouts

Grouts are frequently used in repair and strengthening work to fill voids or to close the space between adjacent portions of concrete. Many types of grouts are available and the proper grouts must be chosen for intended usage.

Conventional grout consist of cement , sand and water and is proportioned to provide a very fluid mix which can be poured into the space to filled. Forms and closure necessary to contain the liquid grout until it has set. Conventional grout of this type has excessive shrinkage characteristics due to the high volume of water in the mix. Placing grout in a space of 2 cm to 5 cm wide will result in enough shrinkage to form a very visible crack at one side of the grouted space. Thus, conventional grouts should be used only when such cracking due to shrinkage will be acceptable.

Cement milk is formed by mixing cement with water into a fluid to place in the very small cracks. Super plasticizers are required with such mixes to maintain the water at an appropriate quantity required to hydrate the cement.

Non- shrink grouts are available for use when it is desirable to fill a void without the normal shrinkage cracks. The dry ingredients for non-shrink grout comes premixed in sacks from the manufacturer and are mixed with water in accordance with manufacturer's instruction. There are many types of non-shrink grouts available, but designers should be aware that the cost of these materials is considerably more than that of conventional grout. The properties of mixed with these materials should be known before specifying their use on a repair or strengthening project.

Epoxy or resin grouts are also available for conditions when high shear force or positive bonding is necessary across a void. Epoxy grouts come prepackaged from the manufacturer and must be mixed and used in strict accord to the instruction. Placement must be completed within the pot life of the resin before the ingredients have set. Epoxy grout generally does not shrink and provides a bonding similar to that of epoxy products. (fig:2.3)

Many other types of grouts can be created using polymer products and other newer concrete products. Shrinkage of conventional grout can be reduced using super plasticizers. The designer should become thoroughly familiar with the properties of the materials which are to be used on his project, and trail batch should be mixed and tested where appropriate.

Injection of grouts required special equipment and specially trained personnel .this method is used to repair of the members that are compressed by filling the joints, cracks, or gaps. It is also used in the restoration of the bearing surface or footing.(fig:2.4)

In many instances, it is inappropriate to fill a void with a fluid grout and a dry material that is packed or tamped into the void is used. Such a material is called a dry pack and consists of cement and sand with only a slight bit of water to moisten the dry ingredient. Dry pack is placed in the void and hand tamped with the rod until the void is filled. Dry pack should be used only in sizable voids which are wide enough to allow through compaction by tamping. Due to its low water content, dry pack generally has low shrinkage properties.

Figure 2.3 Epoxy grouting machine (Source: MRB & Associates)

Figure 2.4 Grouting on weak column (Source: MRB & Associates)

2.3.1.4. Resin concretes

In resin based concrete mixes, the cement is replaced by two component system, one component being based on liquid resin (epoxy, polyester, polyurethane, acrylic, etc.), which will react by cross linking with the second component, called hardener. Resin concrete can be useful in patching small spalled areas of concrete and are not in general use for large volumes of new concrete. Resin concretes require not only a special aggregate mix to produce the desired properties but also special working conditions, since all two component systems are sensitive to humidity and temperature.

The properties of resin concrete are as various as the number of resins offered by the industry for this purpose. However, there are some common tendencies of this relatively new construction material that should especially be taken into consideration, when using it for repair and/or strengthening works:

- Resin has a pot life which must be strictly adhered to in use so that the work is complete before the resin hardens.
- For the resin types used for construction purposes, normal reaction cannot be reached at low temperature (below +10° c); in warm weather the heat developing during the reaction can be excessive and give rise to an excessive shrinkage of the mix.
- Although the direct bond of a resin compound on a clean and dry concrete surface is excellent, a resin concrete has generally poor direct bond on concrete, due to the fact that there can only be a point to point connection between the resin covered aggregates and the old concrete. Thus, to assure a good bond it is necessary to apply a first coating of pure liquid resin onto the existing concrete surface.
- Resin concrete will commonly have a much higher strength but also a different elasticity than normal concrete; problems resulting from the different elasticity must be appropriately considered

The designer should use resin concretes only after a thorough investigation of the properties and material limitation with the existing building materials.

2.3.1.5. Polymer Modified Concrete

Polymer modified concrete is produced by replacing part of conventional cement with certain polymers which are used as cementitious modifiers. The polymer which are normally supplied as dispersions in water, act in several ways. By functioning as water reducing plasticizer they can produce a concrete with better workability, lower water-cement ratio and lower shrinkage elements. They act as integral curing aids, reducing but not eliminating the need for effective curing. By introducing plastic links into the binding system of the concrete, they improve the strength of the harden concrete. They can also increase the resistance of the concrete to some chemical attacks. However, it must be cautioned that such polymer modified concretes are bound to lose all additional properties in case they come under fire. Their alkalinity and, thus, the resistance against carbonating will be much inferior to normal concrete. The design should use polymer modified concrete only after a thorough investigation of the properties for compatibility with the existing building materials.

2.3.1.6. Fiber or reinforced polymers (FRP and CFRP)

Fiber reinforced composite materials are blends of a high strength, high modulus fiber with a hardenable liquid matrix. In this form, both fiber and matrix retain their physical and chemical identities and gives combination properties that cannot be achieved with either of the constituents acting alone. The fibers are highly directional, resulting behavior much like steel reinforced concrete. This behavior of fiber gives designer freedom to tailor the strengthening system to reinforce specific stresses. FRP material properties includes low specific gravity, high strength to weight ratio, high modulus to weight ratio, low density, high fatigue strength, high wear resistance, vibration absorption, dimensional stability, high thermal and chemical stability. Also, FRP materials are very resistance to corrosion. Characteristic of FRP material is the almost linear to elastic stress- strain curve to failure.

FRP materials are very much suitable for repair and strengthening process, especially for seismic loading. Wrapping FRP sheet with epoxy resin around the column upgrades its ductility due to increase in shear strength.

Pre- treatment shall be made on the surface of the column to be wrapped with carbon fiber sheet. The corner cross section of column shall be rounded with the corner radius of 20 mm or larger. This rounded portion must be straight and uncurved along the column height. While wrapping, the fiber direction shall be perpendicular to the column axis and column shall be securely and tightly wrapped with FRP sheet. Overlap of FRP sheet shall be long enough to ensure the rupture in material, lap length shall not be less than 200 mm.

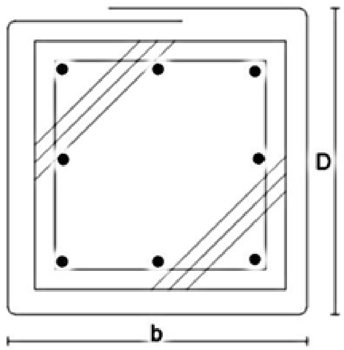


Figure 2.5 Cross- section of Column

FRP sheet shall be wrapped around the column. Position of lap splice shall be provided alternately. Impregnate adhesive resin shall be the one which has appropriate properties in construction and strength to bring the strength characteristic of FRP. After impregnation of adhesive resin has completed the initial hardening process, mortar, boards, or painting must be provided, for fire resistance, surface protection or design point of view.

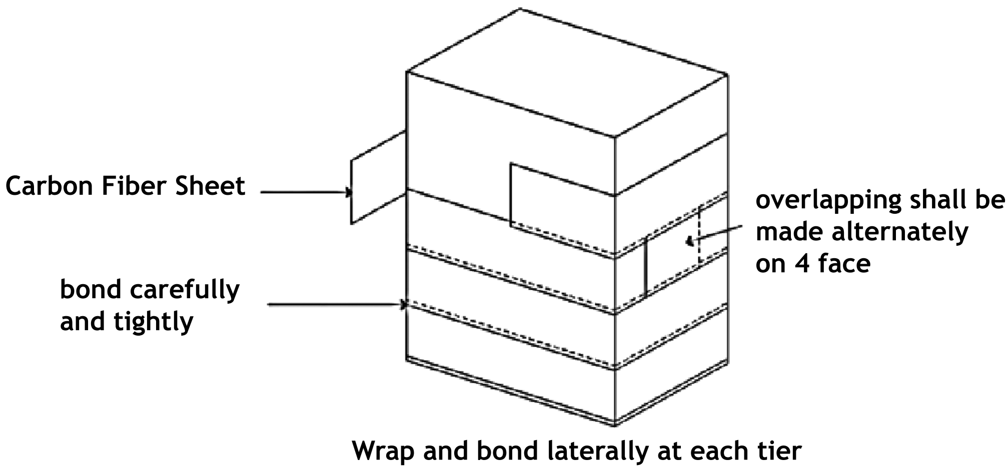


Figure 2.6 Column retrofit

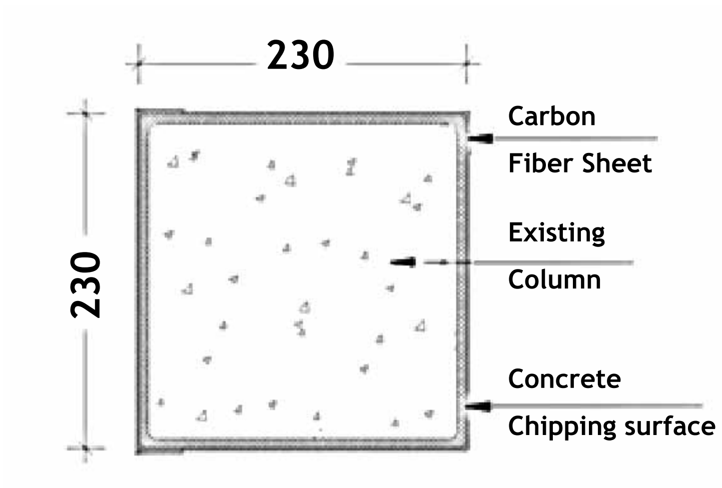


Figure 2.7 Plan view of column retrofit using carbon fiber sheet

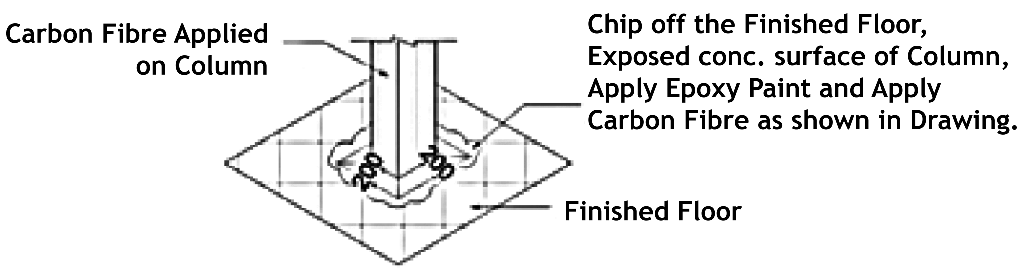


Figure 2.8 3D view of carbon fiber applied in column at floor level

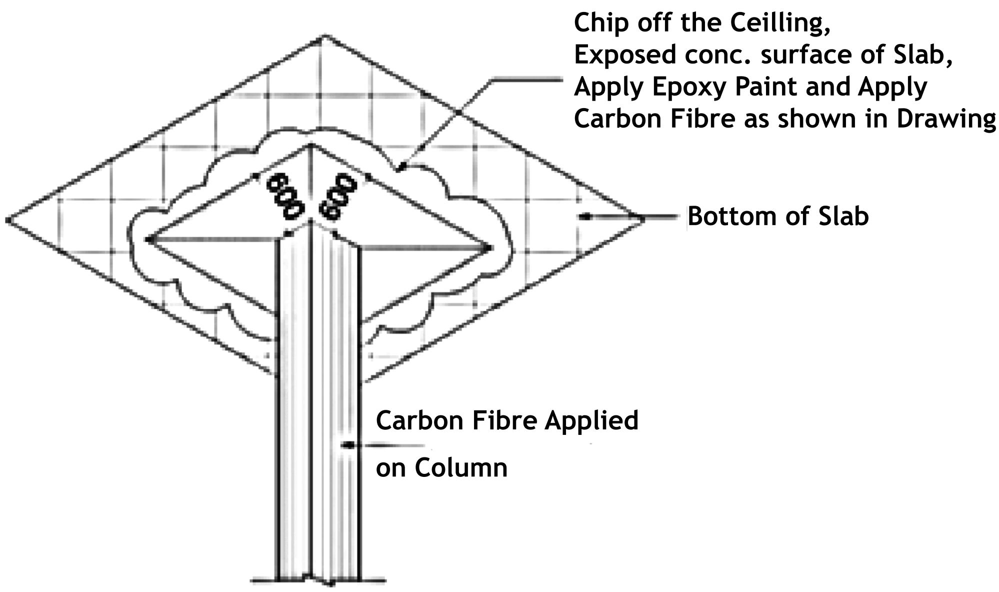


Figure 2.9 3D view of carbon fiber applied in column at ceiling level

The flowchart showing the construction process with continuous fiber sheet is illustrated in Figure 2-10.

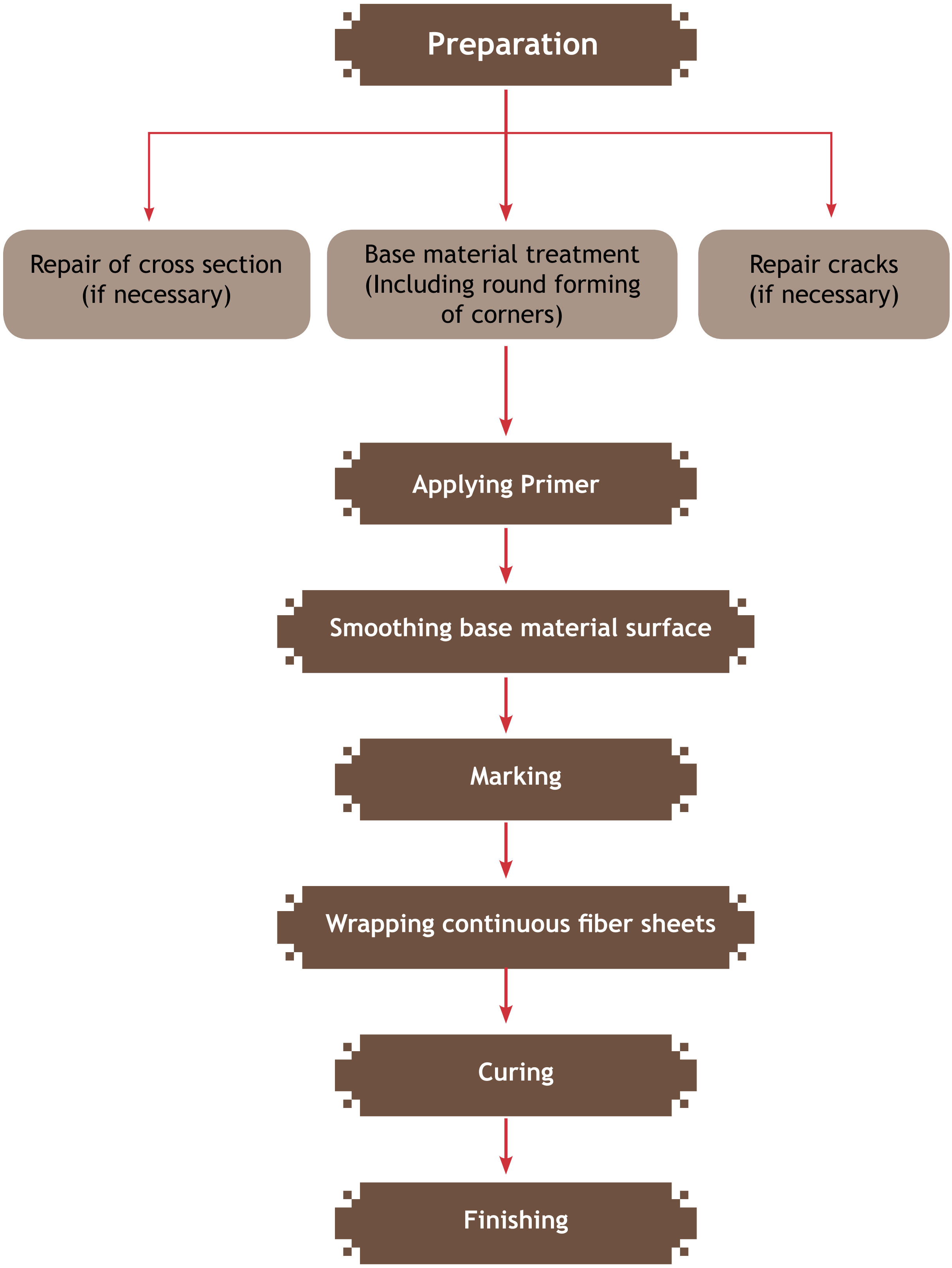


Figure 2.10. Construction process with continuous fiber sheet

3. REQUIRED PERFORMANCE LEVEL

3.1. FOR STRUCTURAL ELEMENTS

Limiting damage condition which may be considered satisfactory for a given building and given ground motion can be described as performance level.

The limiting condition is described by the physical damage within the building, the threat to life safety of the building's occupants created by the damage, and the post-earthquake serviceability of the building. The performance level ranges are assigned as:

3.1.1. IMMEDIATE OCCUPANCY (IO)

The post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral force resisting systems of the building retain nearly all of their pre- earthquake characteristic and capacities.

The risk of the life –threatening injury from structural failure is negligible, and the building should be safe for unlimited egress, ingress, and occupancy.

3.1.2. LIFE SAFETY (LS)

The post -earthquake damage state in which significant damage to the structure may have occurred but in which some margin against either total or partial structural collapse remains. Major structural components have not become dislodged and fallen, threatening life safety either within or outside the building. While injuries during the earthquake may occur, the risk of life threatening injury from structural damage is very low. It should be expected that extensive structural repairs will likely be necessary prior to reoccupation of the building.

3.1.3. COLLAPSE PREVENTION (CP)

This level is the limiting post-earthquake structural damage state in which the building's structural system is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral force resisting system. Although the building retain its overall stability, significant risk of injury due to falling hazards may exist both within and outside the building and significant aftershock may lead to collapse. It should be expected that significant major structural repair will be necessary prior to re-occupancy.

3.2. FOR NON STRUCTURAL ELEMENTS

3.2.1. IMMEDIATE OCCUPANCY (IO)

The post-earthquake damage state in which nonstructural elements and systems are generally in place and functional. Although minor disruption and cleanup should be expected, all equipment and machinery should be working. Contingency plans to deal with possible difficulties with external communication, transportation and availability of supplies should be in place.

3.2.2. LIFE SAFETY (LS)

The post-earthquake damage state could include minor disruption and considerable damage to nonstructural components and system particularly due to damage or shifting of contents. Although equipment and machinery are generally anchored or braced, their ability to function after strong shaking is not considered and some limitations on use or functionality may exist. Standard hazard from breaks in high pressure, toxic or fire suppression piping should not be present. While injuries during the earthquake may occur, the risk of life threatening injuries from nonstructural damage is very low.

3.2.2. COLLAPSE PREVENTION (CP)

This post-earthquake damage state could include extensive damage to nonstructural components or systems but should not include collapse or falling of large and heavy items that could cause significant injuries to group of people, such as parapets, masonry exterior walls, cladding or large heavy ceilings. Nonstructural systems, equipments and machinery may not be functional without replacement or repair. While isolated serious injuries could occur, risk of failures that could put large numbers of people at risk within or outside the building is very low.

4. SEISMIC ASSESSMENT

4.1. RAPID ASSESSMENT (VISUAL SURVEY)

Rapid Seismic Assessment is the preliminary assessment, which concludes the recent status of the building as is it is suitable to live in or not, can be retrofitted or not. In this process, the first level is site inspection, which is also called as visual survey.

4.1.1. METHODOLOGY FOR RAPID SEISMIC ASSESSMENT

1. Review available Structural and Architectural Drawings
2. Review of the Design Data. if available.
3. Interview with the Designer, if possible.
4. Inspection of the Buildings.
5. Identification of Vulnerability Factors as per FEMA 310.
6. Determination of Strength of the Structural Components using Schmidt Hammer
7. Analysis of the Structural Systems, as per guidelines of FEMA 310.
8. Latest Photographs of the Building

4.1.2. BUILDING – FACTS

1. Age of building
2. Structural System – Load bearing Or Frame Structure
3. Foundation Exploration
4. Load path
5. Geometry
6. Walls Detail – Size and mortar
7. Beam and Column Size
8. Water proofing method
9. Renovation of Building
10. Other Structures added

4.2. PRELIMINARY EVALUATION

A preliminary evaluation of building is carried out which involves broad assessment of its physical condition, robustness, structural integrity and strength of structure, including simple calculations. Approximate and quick checks shall be used to compute the strength and stiffness of building components. The seismic base shear and storey shears for the building shall be computed in accordance with IS1893 (Part1).

The preliminary evaluation is a quick procedure to establish actual structural layout and assess its characteristics that can affect its seismic vulnerability. It is a very approximate procedure based on conservative parameters to identify the potential earthquake risk of a building and can be used to screen buildings for detailed evaluation.

4.2.1. SITE VISIT

A site visit will be conducted by the design professional to verify available existing building data or collect additional data, and to determine the condition of the building and its components.

4.2.2. ACCEPTABILITY CRITERIA

A building is said to be acceptable if it meets all the configuration-related checks as well as global level checks on axial and shear stress as outlined in the following sections.

4.2.3. CONFIGURATION RELATED CHECKS

4.2.3.1. Load Path:

The structure shall contain at least one rational and complete load path for seismic forces from any horizontal direction so that they can transfer all inertial forces in the building to the foundation.

4.2.3.2. Redundancy:

The number of lines of vertical lateral load resisting elements in each principal direction shall be greater than or equal to 2. In the case of moment/braced frames, the number of bays in each line shall be greater than or equal to 2.

4.2.3.3. Geometry:

No change in the horizontal dimension of lateral force resisting system of more than 50% in a storey relative to adjacent stories, excluding penthouses and mezzanine floors, should be made.

4.2.3.4. Weak Storey:

The strength of the vertical lateral force resisting system in any storey shall not be less than 70% of the strength in an adjacent storey.

4.2.3.5. Soft Storey:

It is one in which the lateral stiffness is less than70 percent of that in the storey above or less than80 percent of the average lateral stiffness of the three storeys above.

4.2.3.6. Vertical Discontinuities:

All vertical elements in the lateral force resisting system shall be continuous from the root to the foundation.

4.2.3.7. Mass:

There shall be no change in effective mass more than 100% from one storey to the next. Light roofs, penthouses, and mezzanine floors need not be considered.

4.2.3.8. Torson:

The estimated distance between a storey center of mass and the storey centre of stiffness shall be less than 30% of the building dimension at right angles to the direction of loading considered.

4.2.3.9. Adjacent Buildings:

The clear horizontal distance between the building under consideration and any adjacent building shall be greater than 4% of the height of the shorter building, except for buildings that are of the same height with floors located at the same levels.

4.2.3.10. Short Columns:

The reduced height of a column due to surrounding parapet, infill wall, etc. shall not be less than five times the dimension of the column in the direction of parapet, in fill wall, etc. or 50% of the nominal height of the typical columns in that storey.

4.2.3.11. Strength-Related Checks

Approximate and quick checks shall be used to compute the strength and stiffness of building components. The seismic base shear and storey shears for the building shall be computed in accordance with IS 1893 (Part1) and NBC 105(Table 41 and Table 42).

	IS 1893:2002	NBC105:1994
The design horizontal seismic coefficient	$A_h = \frac{ZIS_a}{2R_g}$ Where, Z = Zone factor I = Importance factor R = Response reduction factor $\frac{S_a}{g}$ = Average response accelerationcoefficient	C_d = CZIK Where, C = Basic seismic coefficient Z= Seismic zoning factor K = Structural performance factor
The total design lateral force or design seismic base shear (VB) along any principal direction is determined by using the expression	$V_B = A_h W$ Where, W = Seismic weight of the building IS 1893	$V = C_d W_t$ NBC 105
The approximate fundamental natural period of vibration (T_a) in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:	$T_a = \frac{0.09h}{\sqrt{d}}$ Where, h = Height of Building in meter d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force	$T_a = \frac{0.09h}{\sqrt{d}}$

Table 4.1 Calculation of earthquake load using seismic coefficient method

	IS 1893:2002	NBC 105:1994
The design base shear (V_B) shall be distributed along the height of the building as per the expression	$Q_i = V_B \frac{W_i h_i^2}{\sum W_i h_i^2}$ Where, Q_i= Design lateral force at floor i , W_i = Seismic weight of floor i , h_i = height of floor i , from the base	$F_i = V \frac{W_i h_i}{\sum W_i h_i}$ Where, F_i = Horizontal seismic force at floor i

Table 4.2 Distribution of base shear and calculation of shear stress in RC columns

(a) Shear stress in RC frame columns

Average shearing stress in column is given as,

$$T_{col}=\left(\frac{n_c}{n_c-n_f}\right)\times\left(\frac{V_c}{A_c}\right) < \text{min. of } 0.4 \text{ Mpa and } 0.1\sqrt{f_{ck}}$$ (Ref IS 15988:2013 Clause 6.5.1 b)

For Ground Storey columns,

n_c = Total no. of columns resisting lateral forces in the direction of loading

n_f = Total no. of frames in the direction of loading

A_c = Summation of the cross – section area of all columns in the storey under consideration

V_j = Maximum Storey shear at storey level 'j'

(b) Axial Stress Check:

Axial stresses due to overturning forces as per FEMA 310 (clause 3.5.3.6)

Axial stress in moment frames

Axial force in columns of moment frames at base due to overturning forces,

The axial stress of columns subjected to overturning forces F_o is given by

$$F_o=\frac{2}{3}\left(\frac{V_b}{n_f}\right)\times\left(\frac{H}{L}\right)$$ (Ref IS:15988:2009 clause 6.5.4)

V_b = Base shear x Load Factor

H=total height

L=Length of the building

4.3. DETAILED EVALUATION

A detailed evaluation is required unless results of preliminary evaluation are acceptable. The detailed evaluation procedure is based on determining the probable strength of lateral load resisting elements and comparing them with the expected seismic demands.

4.3.1. CONDITION OF THE BUILDING COMPONENTS

The building should be checked for the existence of some of the following common indicators of deficiency.

4.3.1.1. Deterioration of Concrete

There should be no visible deterioration of the concrete or reinforcing steel in any of the vertical or lateral force resisting elements.

4.3.1.2. Cracks in Boundary Columns

There shall be no existing diagonal cracks wider than 3 mm in concrete columns that encase masonry infills.

4.3.1.3. Masonry Units

There shall be no visible deterioration of masonry units.

4.3.1.4. Masonry Joints

The mortar shall not be easily scraped away from the joints by hand with a metal tool, and there shall be no areas of eroded mortar.

4.3.1.5. Cracks in Infill Walls

There shall be no existing diagonal cracks in infill walls that extend throughout a panel, are greater than 3 mm, or have out-of-plane offsets in the bed joint greater than 3 mm.

4.3.1.6. Condition of the Building Materials

An evaluation of the present day strength of materials can be performed using on-site non-destructive testing and laboratory analysis of samples taken from the building. Field tests are usually indicative tests and therefore should be supplemented with proper laboratory facilities for accurate quantitative results.

Figure 4.1Schmidt Hammer (Source: MRB & Associates)

Figure 4.2 Ferro- scanner (Source: MRB & Associates)

Figure 4.3 Ultra sonic range finder (source: MRB & Associates)

4.4. EVALUATION PROCEDURE

- *Calculation of Base Shear as defined in Preliminary Evaluation*
- *Shear capacity of frame members shall be adequate to develop the moment capacity at the ends, and shall be in accordance with provisions of IS: 13920 for shear design of beams and columns.*

The design shear force for columns shall be the maximum of:
a) Calculated factored shear force as per analysis,

b) A factored shear force given by,

$$V_u = 1.4 \times \frac{M_1 + m_1'}{h_{st}} \quad \text{(Ref IS 13920:1993 Clause 7.3.4)}$$

M_1 and m'_1 are moment of resistance, of opposite signs, of beams framing into the column from opposite faces

- *All concrete columns shall be anchored into the foundation.*
- *The sum of the moment of resistance of the columns (∑Mc) shall be at least 1.1 times the sum of the moment of resistance of the beams (∑Mb) at each frame joint.*

$$\sum Mc \geq 1.1 \sum Mb \text{(Ref IS15988:2013 Clause7.4.1(c))}$$

Seismic Evaluation

- Site Visit and collection of data.
- **Configuration- Related checks:**
  Load path, geometry, redundancy, weak/ soft storey, mezzanines, vertical discontinuities, mass irregularity, torsion, adjacent buildings and short columns.

Seismic preliminary evaluation

- Calculation of base shear.
- Calculation of shear stress in RC columns.
- Calculation of shear capacity of column.
- Calculation of axial stress in moment- frame columns.

Acceptability criteria Satisfied ?

Detailed evaluation

- Calculation of moment of resistance in hogging and sagging.
- Check of shear capacity of Beam and Column.
- Check of strong column/ weak beam.

Acceptability criteria Satisfied ?

Strengthening not recommended.

Strengthening recommended.

Selection and design of retrofit strategies

Comparison of various retrofitting option with reference to:

- Cost
- Time consuming
- Disturbance to existing structure
- Effect as original aesthetics

Selection of most appropriate retrofitting option

- Detail drawing & report
- Relatively Safe Area drawing

Selection of most appropriate retrofitting option

5. CATEGORIZATION OF DAMAGE GRADE

5.1. Damage Categorization Table

S.N.	Damage Grades	Level of Damage	Recommendations after Earthquake	Remarks
G1	Negligible – slight damage (Non or slight structural)	Only thin cracks in some wall plaster, can fall of plaster parts, fall of loose brick or stone from upper parts.	Only architectural repair needed. Appropriate seismic strengthening advised.	

Figure 5.1 Damage grade 1 (source: MRB & Associates)

S.N.	Damage Grades	Level of Damage	Recommendations after Earthquake	Remarks
G2	Moderate damage. (Slight or moderate non-structural damage)	Many thin cracks in walls and in plasters fall of brick or stone work, fall of plaster but no structural part damage.	Only architectural repair needed. Appropriate seismic strengthening advised.	

Figure 5.2 Damage grade 2 (source: MRB & Associates)

S.N.	Damage Grades	Level of Damage	Recommendations after Earthquake	Remarks
G3	Moderate to heavy damage. (Moderate Structure, heavy non structure damage)	Thick and large cracks in many walls, upper structure like tiles or chimney damage failure or non-structural partition wall	Architectural and structural repair required. Grouting in crack advised and strongly advised structure strengthening with technical support.	

Figure 5.3Damage grade 3 (source: MRB & Associates)

S.N.	Damage Grades	Level of Damage	Recommendations after Earthquake	Remarks
G4	Very heavy damage (Heavy structure, very heavy non structure damage)	Large gap occurs in main walls, wall collapses, some structural floor or roof damage.	Immediately vacate the building, demolish and construct with seismic designs. In some case extensive restoration and strengthening can be apply.	Technical Assistance Recommended

Figure 5.4 Damage grade 4 (source: MRB & Associates)

S.N.	Damage Grades	Level of Damage	Recommendations after Earthquake	Remarks
G5	Destruction (Very heavy structure Damage)	Floor collapse due to soft storey, partial or total collapse of building.	Immediately clear the site and reconstruction the building following seismic design.	Technical Assistance Recommended

Figure 5.5 Damage grade 5 (source: MRB & Associates)

6. SEISMIC STRENGTHENING STRATEGY AND SEISMIC RETROFITTING OPTIONS

Seismic strengthening for improved performance in the future earthquakes can be achieved by using one of the several options that will be discussed in this section once an evaluation has been conducted and the presence of unacceptable seismic deficiencies has been detected.
Basic issues that might rise while retrofitting the buildings are:

- **Socio-cultural issues**
  - Heritage sites

- **Economic issues**
  - Cost of demolition & rubble removal
  - Cost of reconstruction
  - Real state
  - Built-up area vs. carpet area

- **Technical issues**
  - Type of structural system
  - Construction materials
  - Site
  - Damage intensity level

- **Legal issues**

  For most buildings and performance objectives, a number of alternative strategies and systems may result in acceptable design solutions. Prior to adopting a particular strategy, the engineer should evaluate a number of alternatives for feasibility and applicability and together with the owner, should select the strategy or combination of strategies that appears to provide the most favorable overall solution.

  The strategies that are discussed in the following stages describe a methodology for the design of the strengthening measures at a general level as modifications to reduce/correct seismic deficiency.

6.1. RETROFIT STRATEGIES

A retrofit strategy is a basic approach adopted to improve the probable seismic performance of a building or otherwise reduce the existing risk to an acceptable level. Strategies relate to modification or control of the basic parameters that affect a buildings earthquake performance. These include the building's stiffness, strength, deformation capacity, and ability to dissipate energy, as well as the strength and character of ground motion transmitted to the building. Strategies can also include combinations of these approaches. For example, the addition of shear walls or braced frames to increase stiffness and strength, the use of confinement jackets to enhance deformability.

There is wide range of retrofit strategies available for reducing the seismic risk inherent in an existing building. These strategies include:

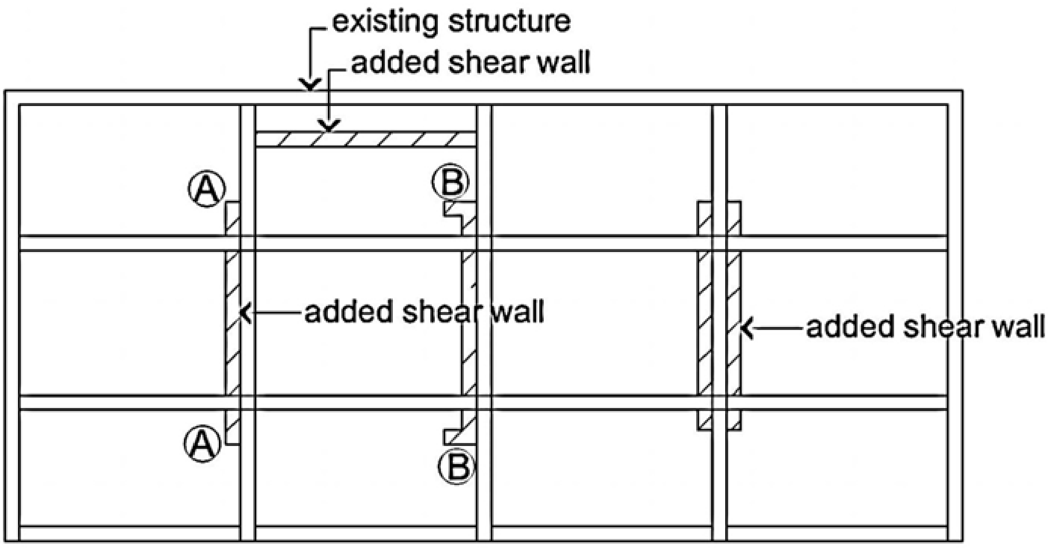
6.1.1. SYSTEM STRENGTHENING AND STIFFENING

System strengthening and stiffening are the most common seismic performance improvement strategies adopted for buildings with inadequate lateral force resisting systems.

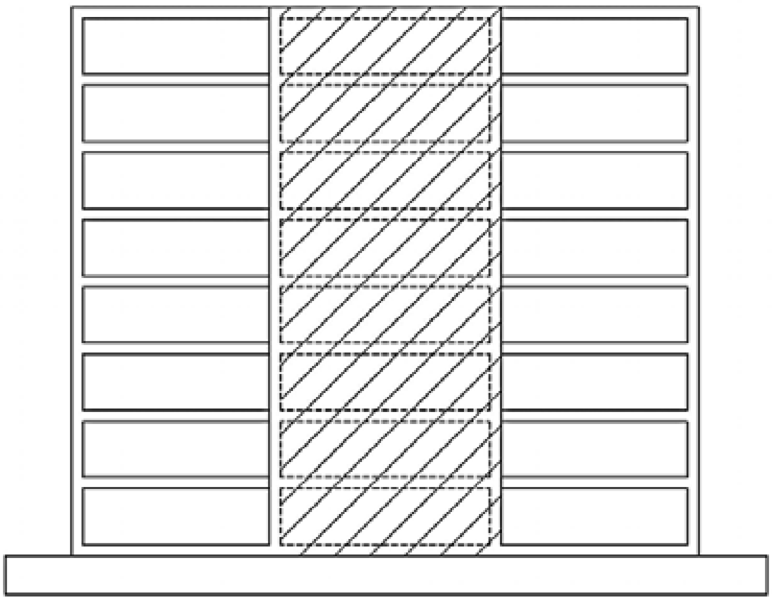
Introduction of new structural elements to the building system can improve the performance of the building. This can be achieved by introducing,

6.1.2. SHEAR WALL INTO AN EXISTING CONCRETE STRUCTURE

The introduction of shear walls into an existing concrete structure is one of the most commonly employed approaches to seismic upgrading. It is an extremely effective method of increasing both building strength and stiffness. A shear wall system is often economical and tends to be readily compatible with most existing concrete structures.



(a)



(b)

Figure 6.1 Shear wall in existing structure (a: Plan, b: Elevation)

6.1.3. BUTTRESSES PERPENDICULAR TO AN EXTERNAL WALL OF THE STRUCTURE

Buttresses are braced frames or shear walls installed perpendicular to an exterior wall of the structure to provide supplemental stiffness and strength. This system is often a convenient one to use when a building must remain occupied during construction, as most of the construction work can be performed on the building exterior, minimizing the convenience to building occupants.

Figure 6.2 Buttress provide to exterior building (source: Hari Darshan Shrestha)

6.1.4. MOMENT RESISTING FRAMES

Moment-resisting frames can be an effective system to add strength to a building without substantially increasing the buildings stiffness. Moment frames have the advantage of being relatively open and therefore can be installed with relatively minimal impact on floor space.

6.1.5. INFILL WALLS

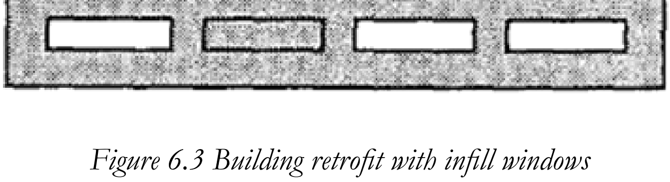


Figure 6.3 Building retrofit with infill windows

6.1.6. TRUSSES AND DIAGONAL BRACES

Braced steel frames are another common method of enhancing an existing buildings stiffness and strength. Typically, braced frames provide lower levels of stiffness and strength than do shear walls, but they add far less mass to the structure than do shear walls, can be constructed with less disruption of the building, result in less loss of light, and have a smaller effect on traffic patterns within the building.

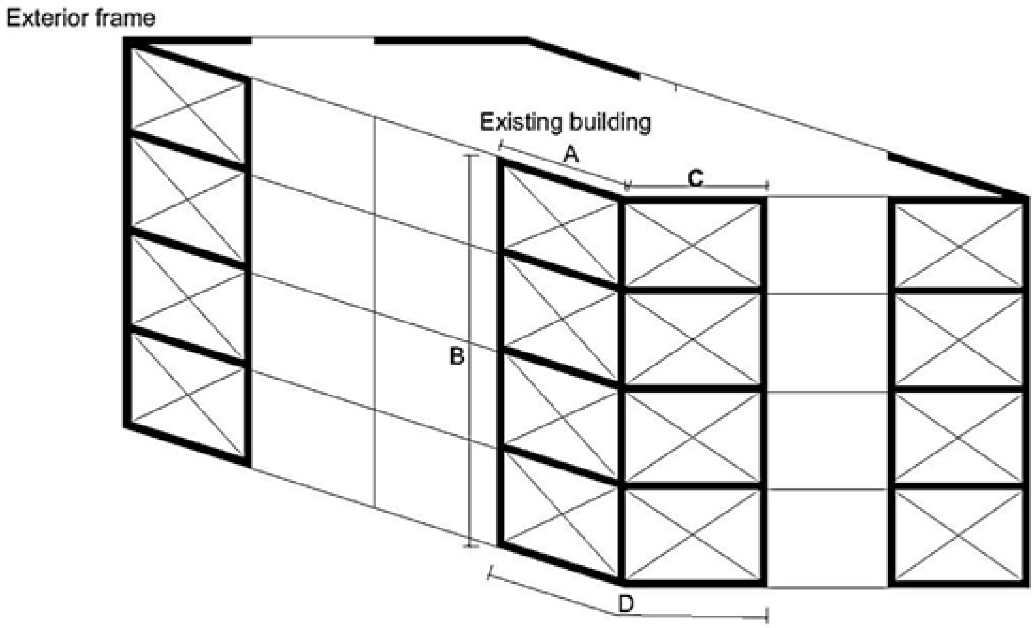


Figure 6.4 Exterior frame (steel framed brace)

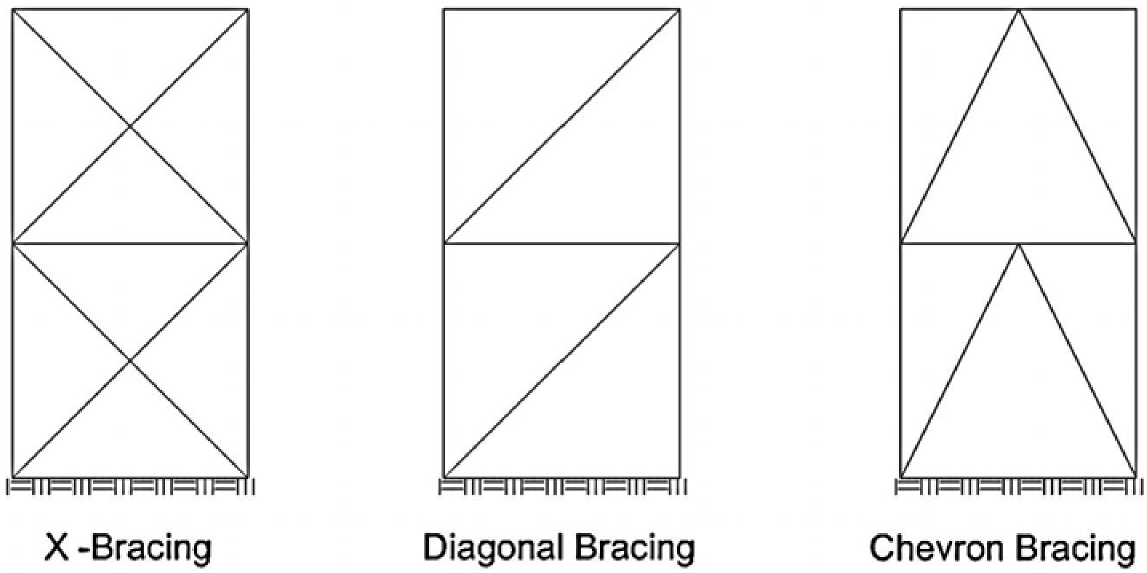


Figure 6.5 Types of bracings

Angle or channel steel profile can be used for the purpose of adding steel braces. Braces should be arranged so that their center line passes through the centers of the beam-column joints.

Likewise, eliminating or reducing structural irregularities can also improve the performance of the building in earthquake such as:

- Vertical irregularities
- Filling of openings in walls
- Pounding effect of the buildings
- Improving diaphragm in the presence of large openings by provision of horizontal bracing.

6.1.7. DIAPHRAGM STRENGTHENING

Most of the concrete buildings have adequate diaphragms except when there occur large openings. Methods of enhancing diaphragms include the provision of topping slabs, metal plates laminated onto the top surface of the slab, or horizontal braced diaphragms beneath the concrete slabs.

6.1.8. STRENGTHENING OF ORIGINAL STRUCTURAL ELEMENTS

Strengthening of reinforced concrete structural elements is one method to increase the earthquake resistance of damaged or undamaged buildings. Repair of reinforced concrete elements is often required after a damaging earthquake to replace lost strength.

Establishing sound bond between the old and the new concrete is of great importance. It can be provided by chipping away the concrete cover of the original member and roughening its surface, by preparing the surface with glues (as epoxy prior concreting), by additional welding of bent reinforcement bars or by formation of reinforced concrete or steel dowels.

Figure 6.6 Strengthening of original structure (Source: MRB & Associates)

Strengthening of original structural elements includes strengthening of:

• ***Columns***

The damage of reinforced concrete columns without a structural collapse will vary, such as a slight crack (horizontal or diagonal) without crushing in concrete or damage in reinforcement, superficial damage in the concrete without damage in reinforcement, crushing of the concrete, buckling of reinforcement, or rupture of ties. Based on the degree of damage, techniques such as injections, removal and replaced or jacketing can be provided. Column jacketing can be reinforced concrete jacketing, steel profile jacketing, steel encasement.

The main purpose of column retrofitting is to increase column flexure and shear strength, improving ductility and rearrangement of the column stiffness.

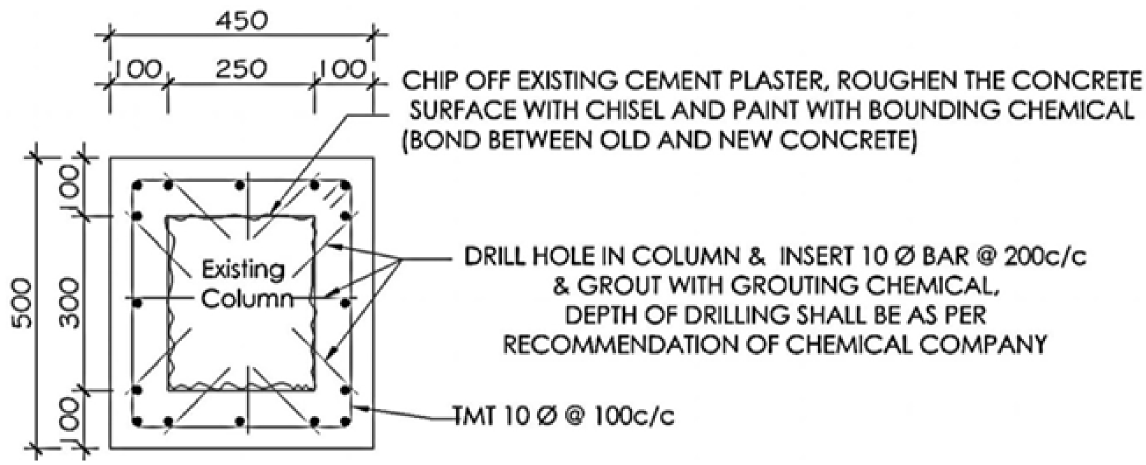


Figure 6.7 Column RC jacketing plan

Figure 6.8 Jacketing of column (source MRB & Associates)

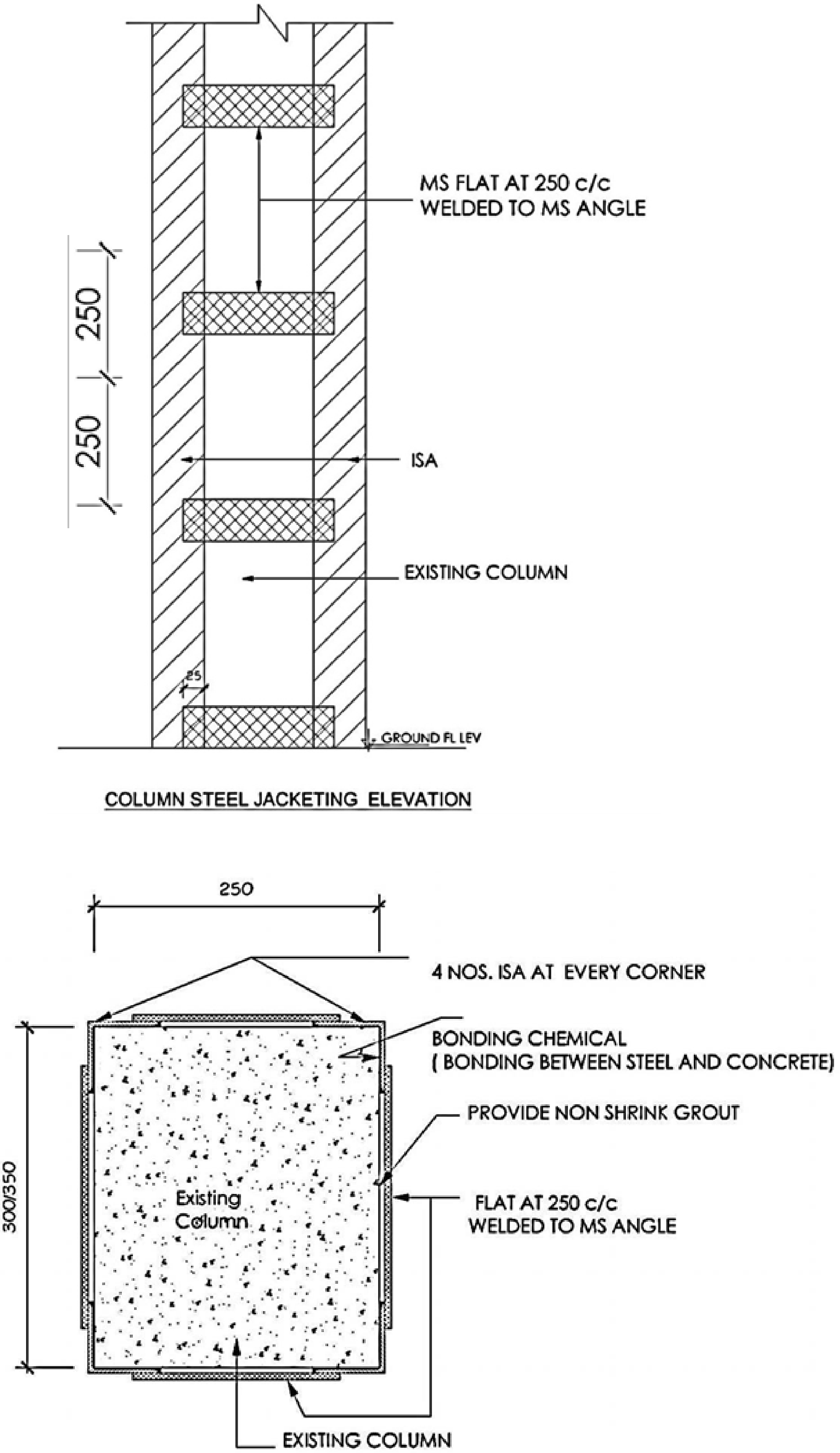


Figure 6.9 Column steel jacketing (a: Elevation, b: plan)

Figure 6.10 Steel jacketing of columns (source: MRB & Associates)

• Beams

The aim of strengthening of beams is to provide adequate strength and stiffness of damaged or undamaged beam which are deficit to resist gravity and seismic loads. It is very important that the rehabilitation procedure chosen provides proper strength and stiffness of the beams in relation to adjacent columns in order to avoid creating structures of the "strong beam weak column" type which tend to force seismic hinging and distress into the column, which must also support major gravity loads.

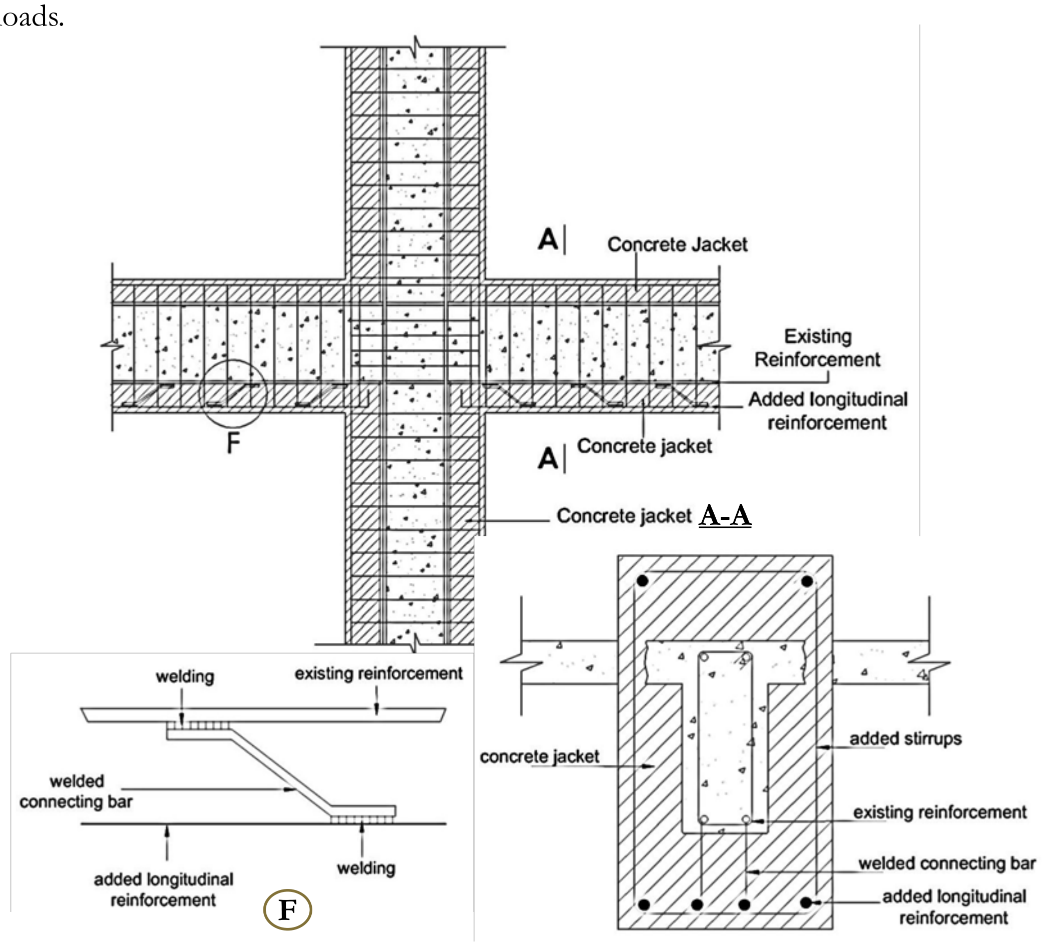


Figure 6.11 RC Jacketing of beam

Figure 6.12 Reinforcement placing for beam jacketing (source MRB & Associates)

Figure 6.13 Top reinforcement detailing of beam jacketing (Source MRB & Associates)

Figure 6.14 Beam Jacket (Source: MRB & Associates)

Figure 6.15 Encasement of existing beam (Source: MRB & Associates)

• *Beam-Column Joints*

The most critical region of a moment resisting frame for seismic loading, the beam to the column joint, is undoubtedly the most difficult to strengthen because of the great number of elements assembled at this place and the high stresses this region is subjected to in an earthquake. Under earthquake loading joints suffer shear and/or bond failures.

The retrofitting at the beam column joint can be done using methods like, reinforced concrete jacketing and steel plate reinforcement.

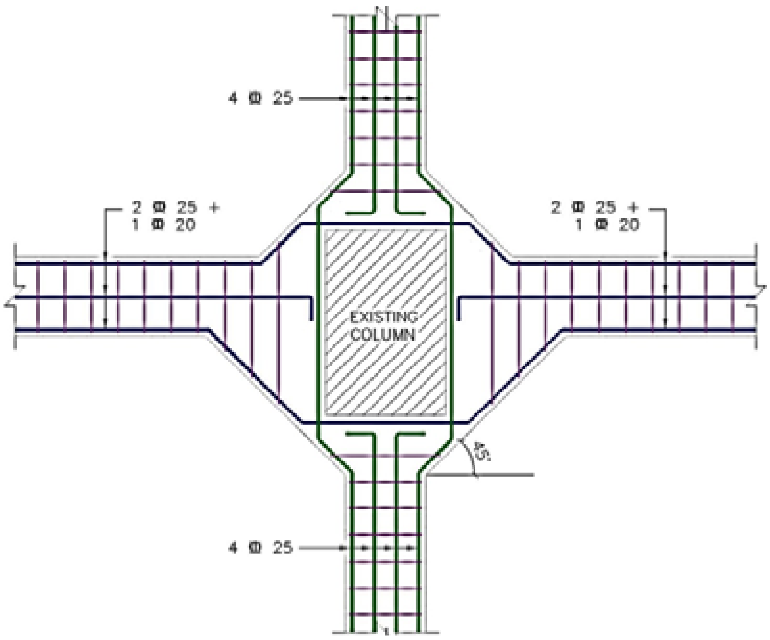
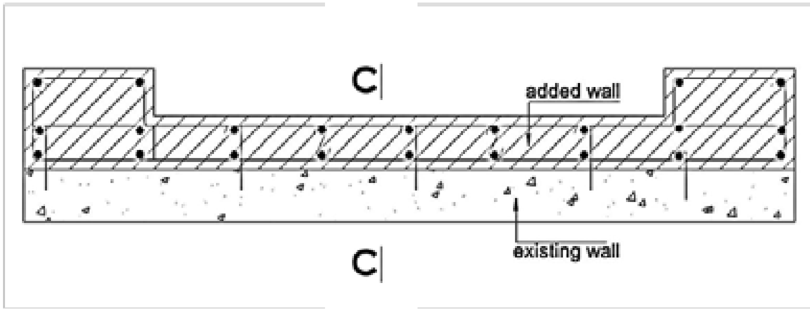


Figure 6.16 Example of beam column joint

- ***Concrete Shear wall***

Shear wall possess great stiffness and lateral strength which provides most significant part of the earthquake resistance of the building. Therefore, a severely damaged or a poorly designed shear wall must be repaired or strengthened in order that the structure's strength for seismic force can be significantly improved.



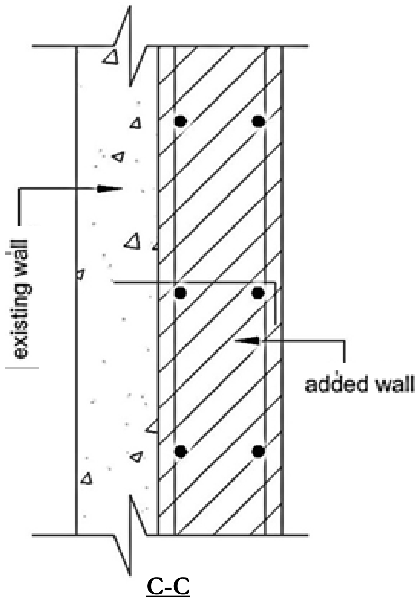


Figure 6.17 Example of shear wall retrofit

- ***Slabs***

Primarily, slabs of floor structures have to carry vertical gravity loads. However, they must also provide diaphragm action and be compatible with all lateral resistant element of the structure. Therefore, slab must possess the necessary strength and stiffness. Damages in slabs generally occur due to large openings, insufficient strength and stiffness, poor detailing, etc.

Strengthening of slab can be done by thickening of slabs in cases of insufficient strength or stiffness. For local repairs, injections should be applied for repair of cracks. Epoxy or cement grout can be used.

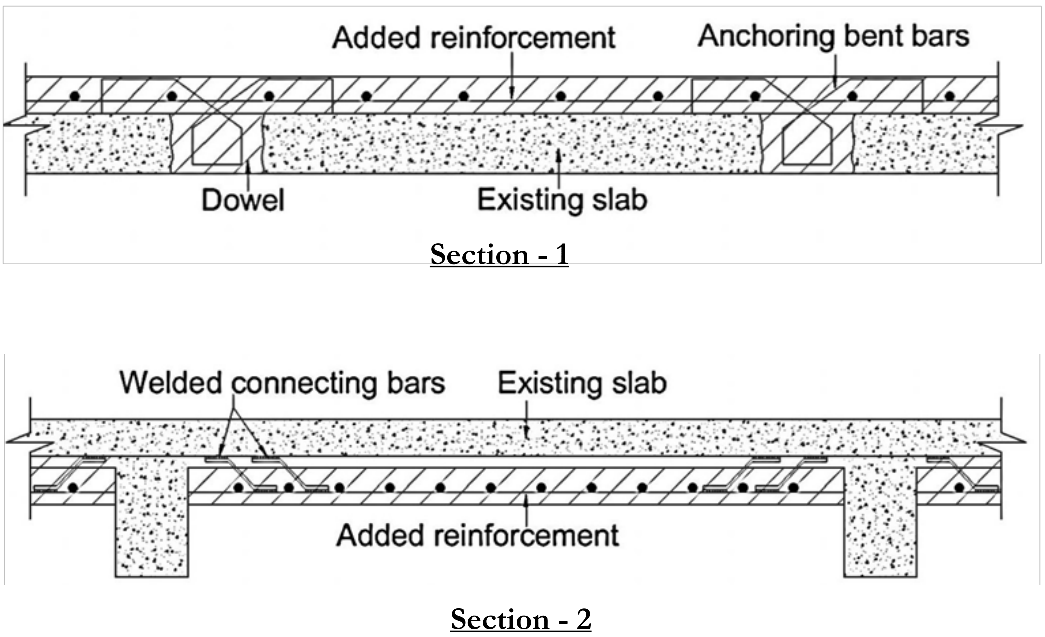


Figure 6.18 Increasing slab thickness

• *Infill Partition wall*

Generally, infilled partition walls in concrete framed buildings are unreinforced although it is highly desirable to be reinforced in seismic region like Nepal. Infilled partition walls in concrete framed buildings often sustain considerable damage in an earthquake as they are relatively stiff and resist lateral forces, often they were not designed to resist, until they crack or fail. Damage may consist of small to large cracks, loose bricks or blocks or an infill leaning sideways. Damage may also result in the concrete frame members and joints which surrounds the infilled wall.

The effect of strengthening an infilled wall must be considered by analysis on the surrounding elements of the structure. Infilled walls are extremely stiff and effective in resisting lateral forces, but all forces must be transferred through the concrete elements surrounding the infilled walls.

• *Foundation*

Retrofitting of foundation is often required when the strength of foundation is insufficient to resist the vertical load of the structure. Strengthening of foundations is difficult and expensive construction procedure. It should be performed in the following cases:

- Excessive settlement of the foundations due to poor soil conditions.
- Damage in the foundation structure caused by seismic overloading.
- Increasing the dead load as a result of the strengthening operations.
- Increasing the seismic loading due to changes in code provisions or the strengthening operations.
- Necessity of additional foundation structure for added floors.

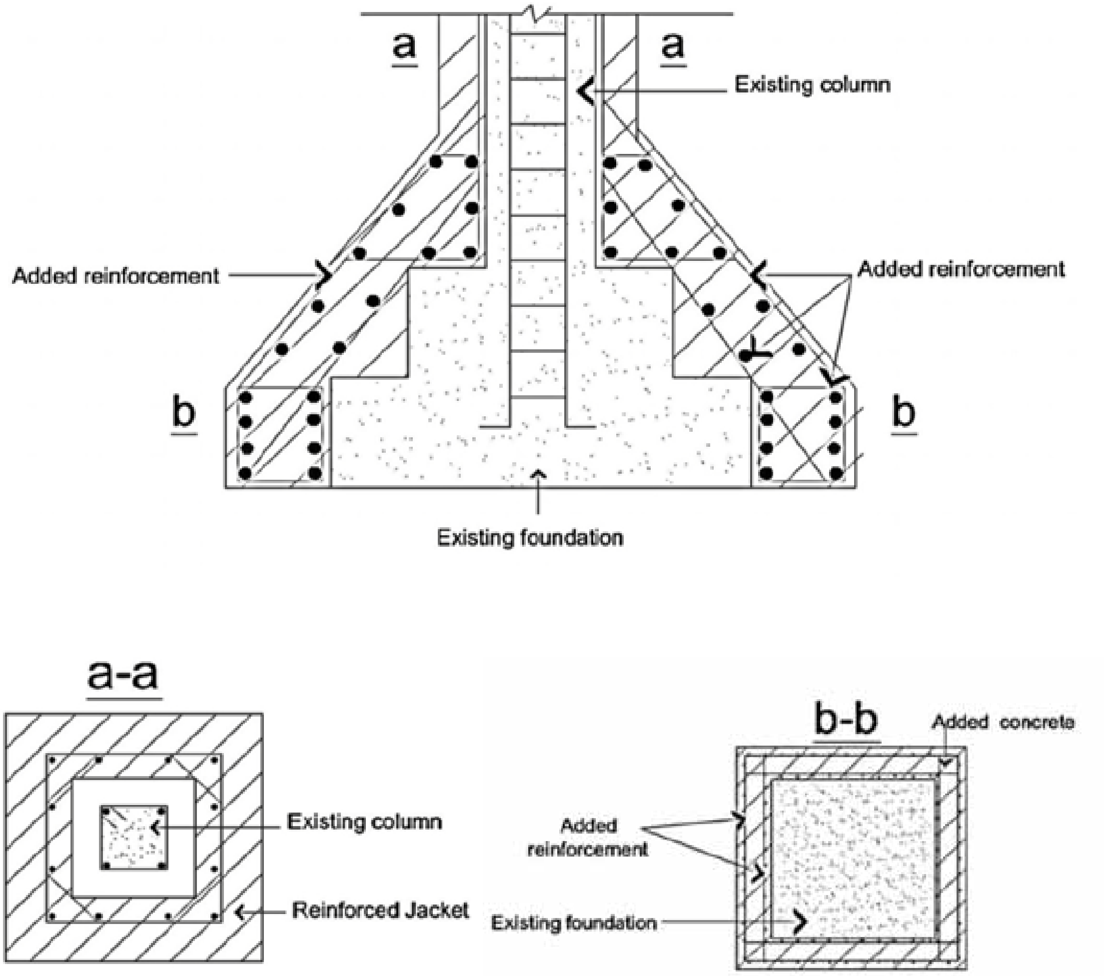


Figure 6.19 Foundation retrofit

Reinforcement layout at foundation for retrofit

Reinforcement layout at foundation for retrofit

Figure 6.20 Reinforcement layout at foundation for retrofit (source: MRB & Associates)

Foundation retrofit with concrete

Reinforcement layout at foundation for retrofit

Figure 6.21Reinforcement layout at foundation for retrofit (source: MRB & Associates)

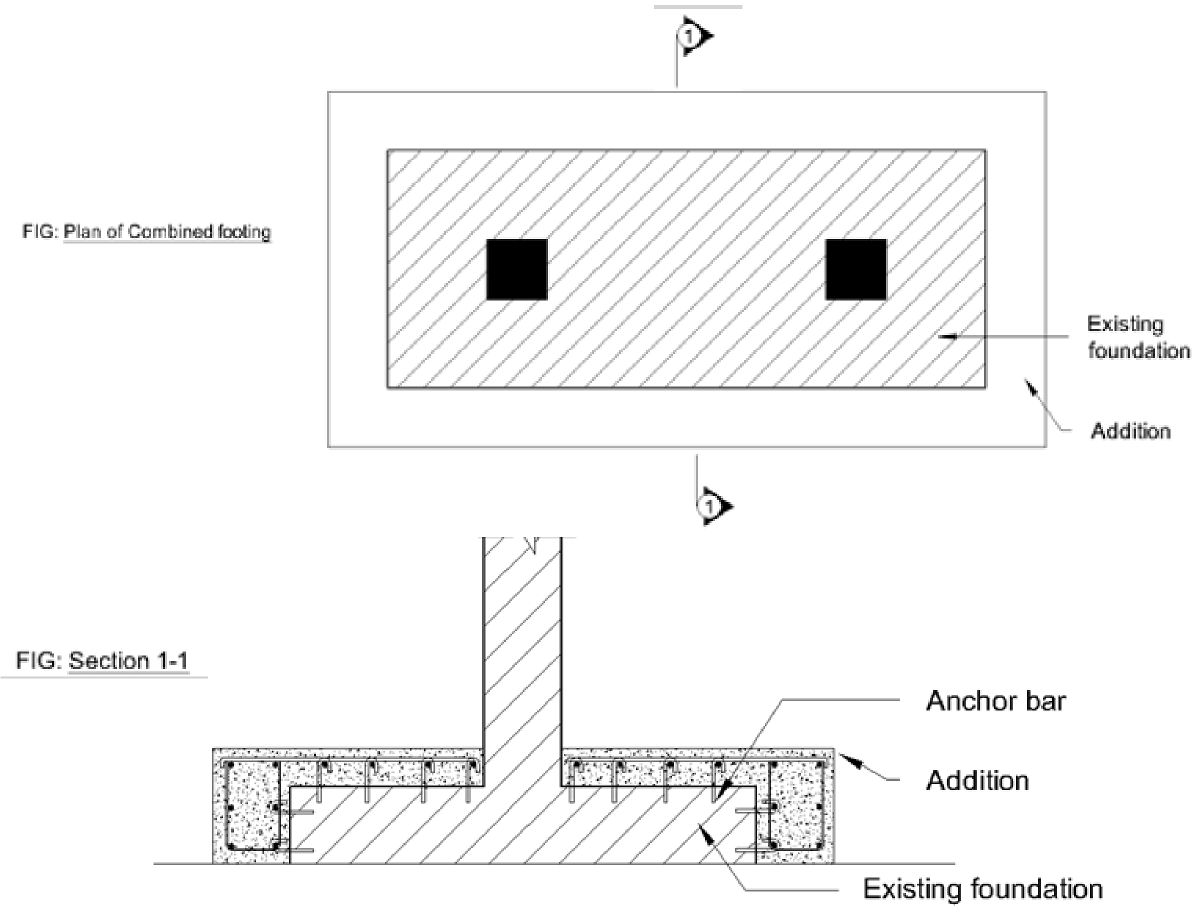


Figure 6.22 Foundation retrofit (Combined)

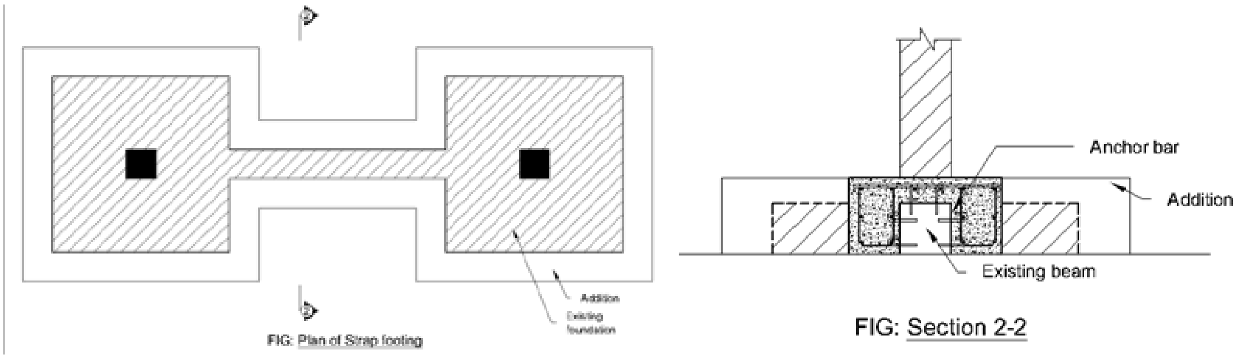


Figure 6.23 Foundation retrofit (Strap)

6.1.9. REDUCING EARTHQUAKE DEMANDS

Rather than modifying the capacity of the building to withstand earthquake-induced forces and deformations, this strategy involves modification of the response of the structure such that the demand forces and deformations are reduced. Irregularities related to distribution of strength, stiffness and mass result poor seismic performance.

The methods for achieving this strategy include reduction in the building's mass and the installation of systems for base isolation and/or energy dissipation. The installation of these special protective systems within a building typically entails a significantly larger investment than do more- conventional approaches. However, these special systems do have the added benefit of providing for reduced demands on building contents.

6.1.9.1. Base Isolation

This approach requires the insertion of compliant bearing within a single level of the building's vertical load carrying system, typically near its base. The bearings are designed to have relatively low stiffness, extensive lateral deformation capacity and may also have superior energy dissipation characteristics. Installation of an isolation system results in a substantial increase in the building's fundamental response period and, potentially, its effective damping. Since the isolation bearings have much greater lateral compliance than does the structure itself, lateral deformation demands produced by the earthquake tend to concentrate in the bearings themselves. Together these effects result in greatly reduced lateral demands on the portion of the building located above the isolation bearings.

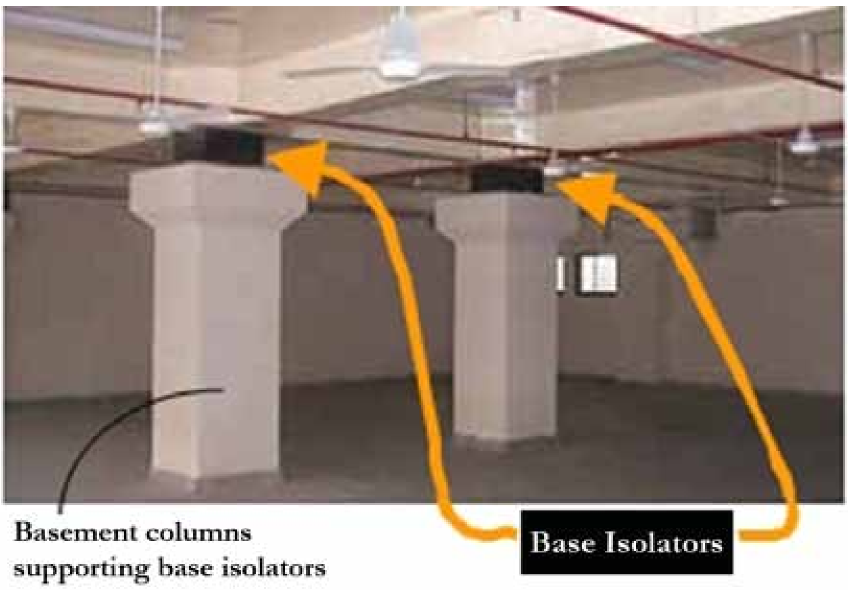


Figure 6.24 Base isolation

6.1.9.2. *Energy Dissipation Systems*

Energy dissipation systems directly increase the ability of the structure to dampen earthquake response in a benign manner, through either viscous or hysteretic damping. This approach requires the installation of energy dissipation units (EDUs) within the lateral force resisting system. The EDUs dissipate energy and in the process reduce the displacement demands on the structure. The installation of EDUs often requires the installation of vertical braced frames to serve as a mounting platform for the units and therefore, typically results in a simultaneous increase in system stiffness. Energy dissipation systems typically have greater cost than conventional systems for stiffening and strengthening a building but have the potential to provide enhanced performance.

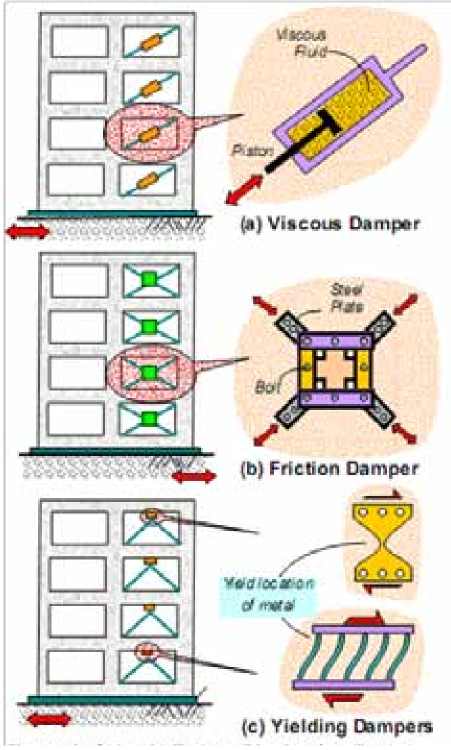


Figure 6.25 Energy dissipation system

6.1.9.3. *Mass Reduction*

The performance of some buildings can be greatly improved by reducing the building mass. Building mass reductions reduce the building's natural period, the amount of inertial forces that develops during its response, and the total displacement demand on the structure.

Mass can be reduced by removing heavy nonstructural elements such as cladding, water tanks, storage, heavy antenna, etc. In the extreme, mass reduction can be attained by removing one or more building stories.

6.2. STRENGTHENING OPTIONS FOR RC FRAMED STRUCTURES

Members requiring strengthening or enhanced ductility can be jacketed by reinforced concrete jacketing, steel profile jacketing, steel encasement or wrapping with FRP's. Depending on the desired earthquake resistance, the level of the damage, the type of the elements and their connections, members can be strengthened by injections, removal and replacement of damaged parts or jacketing.

- RC jacketing involves placement of new longitudinal reinforcement and transverse reinforcement bars in the new concrete overlay around existing member. Perfect confinement by close, adequate shaped stirrups and ties contributes to the improvement of the ductility of the strengthened members.
- Steel profile jacketing can be done through steel angle profiles placed at each corner of the existing reinforced concrete member and connected together as a skeleton with transverse steel straps.
- Another way is by providing steel encasement with thin plates in existing members. Jacketing with steel encasement is implemented by gluing of steel plates on the external surfaces of the original members. The steel plates acting as reinforcement are glued to the concrete by epoxy resin. This technique doesn't require any demolition. It is considerably easy for implementation and there is a negligible increase in the cross section size of the strengthened members.
- Retrofitting using FRPs involves placement of composite material made of continuous fibers with resin impregnation on the outer surface of the RC member.

6.2.1. RC JACKETING OF COLUMNS

Reinforced concrete jacketing improves column flexure strength and ductility. Closely spaced transverse reinforcement provided in the jacket improves the shear strength and ductility of the column. The procedure for reinforced concrete jacketing are:

i. The seismic demand on the columns in terms of axial load (P) and moment (M) is obtained.

ii. The column size and section details are estimated for P and M as determined above.

iii. The existing column size and amount of reinforcement is deducted to obtain the amount of concrete and steel to be provided in the jacket.

iv. Increase the amount of concrete and steel actually to be provided as follows to account for losses.

$A_c = 1.5\ A_c'$ and $A_s = 4/3\ A_s'$(IS:15988:2013Clause 8.5.1.1(e))

Where, A_c and A_s = Actual concrete and steel to be provide in the jacket

A_c' and A_s' = Concrete and steel values obtained for the jacket after deducting the existing concrete and steel from their respective required amount.

v. The spacing of ties to be provided in the jacket in order to avoid flexure shear failure of column and provide adequate confinement to the longitudinal steel along the jacket is given as:

$$s = \frac{f_y dh^2}{\sqrt{f_{ck}} \cdot t_j}$$ (IS:15988:2013Clause 8.5.1.1(f))

Where,

f_y = yield strength of steel

f_{ck} = cube strength of concrete

dh= diameter of stirrup

t_j = thickness of jacket

vi. If the transfer of axial load to new longitudinal steel is not critical then friction present at the interface can be relied on for the shear transfer, which can be enhanced by roughening the old surface.

vii. Dowels which are epoxy grouted and bent into 90° hook can also be employed to improve the anchorage of new concrete jacket.

The minimum specifications for jacketing of columns are:

a. Strength of the new materials must be equal or greater than those of the existing column. Concrete strength should be at least 5MPa greater than the strength of the existing concrete.

b. For columns where extra longitudinal reinforcement is not required, a minimumof 12φ bars in the four corners and ties of8φ @ 100 c/c should be provided with 135° bends and 10φ leg lengths.

c. Minimum jacket thickness should be 100mm.

d. Lateral support to all the longitudinal bars should be provided by ties with an included angle of not more than 135°.

e. Minimum diameter of ties should be 8mm and not less than 1/3 of the longitudinal bar diameter.

f. Vertical spacing of ties shall not exceed200 mm, whereas the spacing close to the joints within a length of ¼ of the clear height should not exceed 100 mm .Preferably, the spacing of ties should not exceed the thickness of the jacket or 200 mm whichever is less.

Option 1:

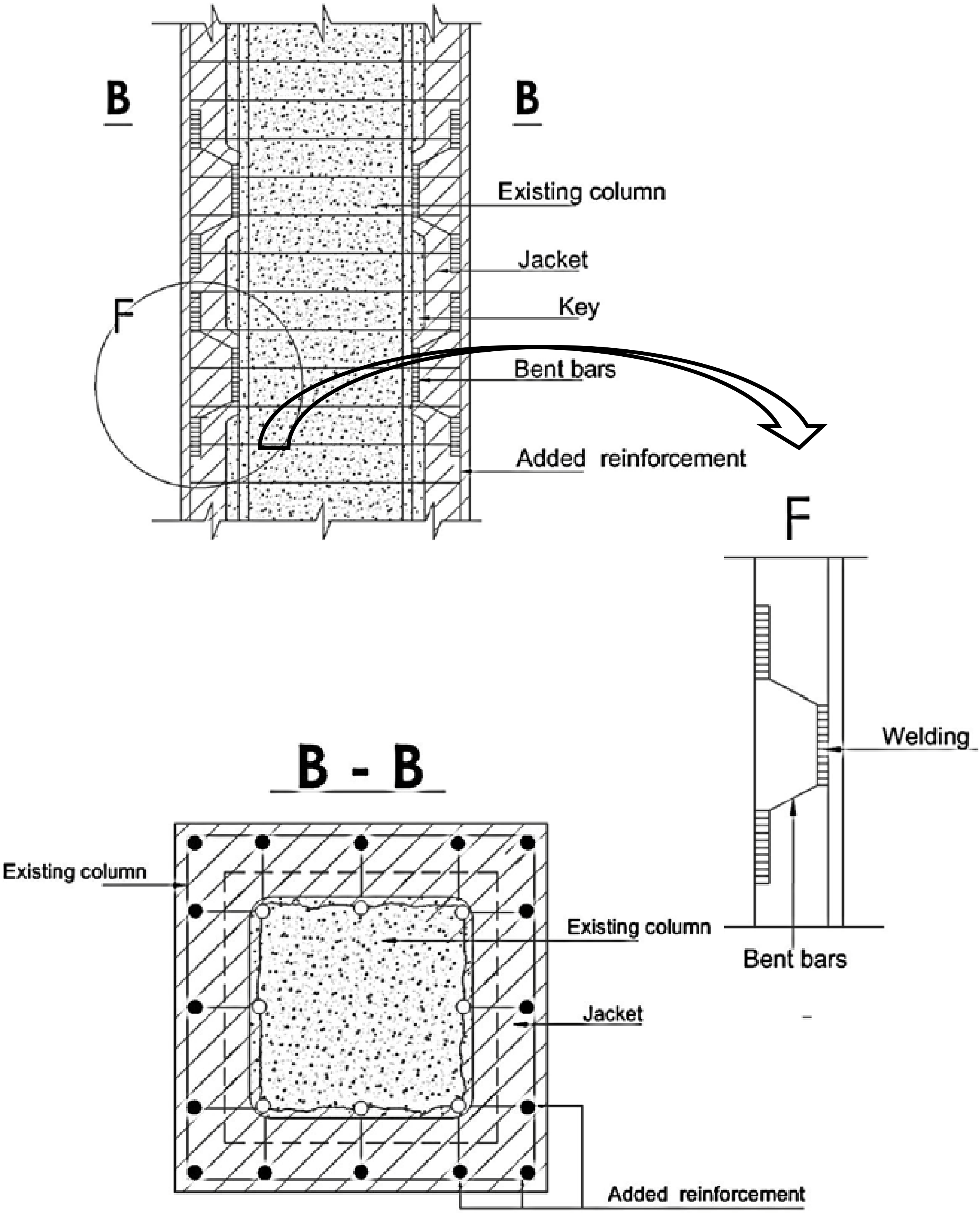
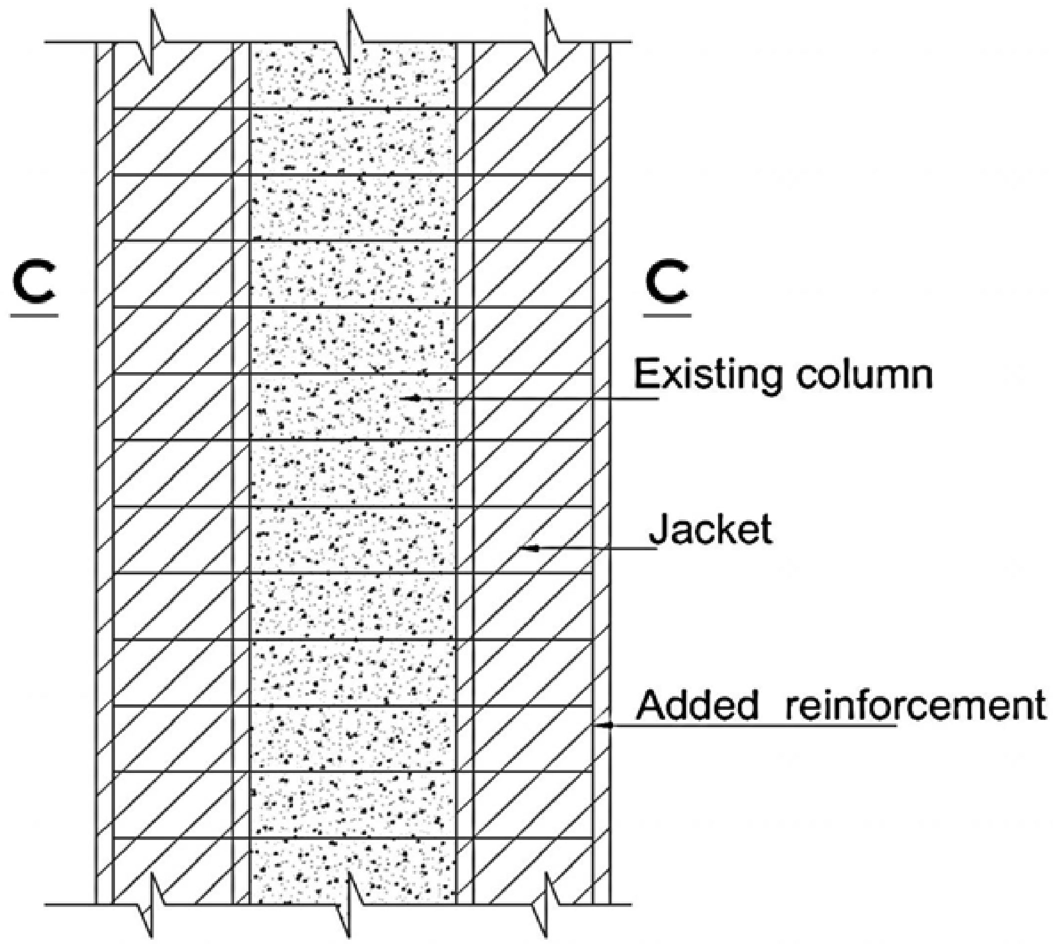


Figure 6.26 Column jacketing with reinforced concrete- option 1

Option 2:



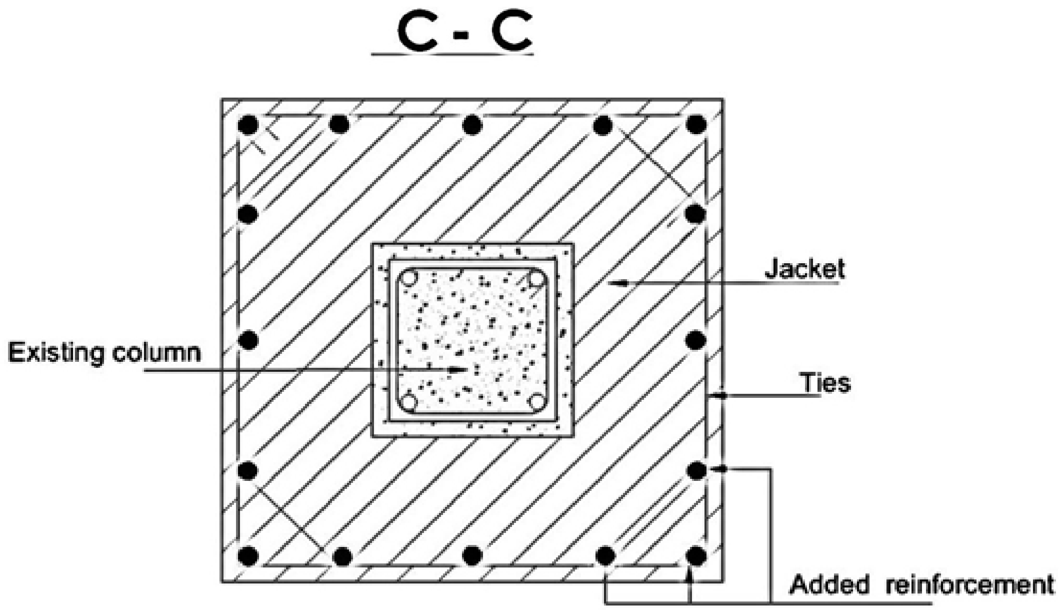


Figure 6.27 Column jacketing with reinforced concrete- option 2

6.2.2. STEEL JACKETING OF COLUMNS

Steel profile skeleton jacketing consists of four longitudinal angle profiles placed one at each corner of the existing reinforced concrete column and connected together in a skeleton with transverse steel straps. They are welded to the angle profiles and can be either round bars or steel straps. The angle profile size should be no less than L 50 $\times$ 50 $\times$ 5. Gaps and voids between the angle profiles and the surface of the existing column must be filled with non-shrinking cement grout or resin grout. In general, an improvement of the ductile behavior and an increase of the axial load capacity of the strengthened column is achieved.

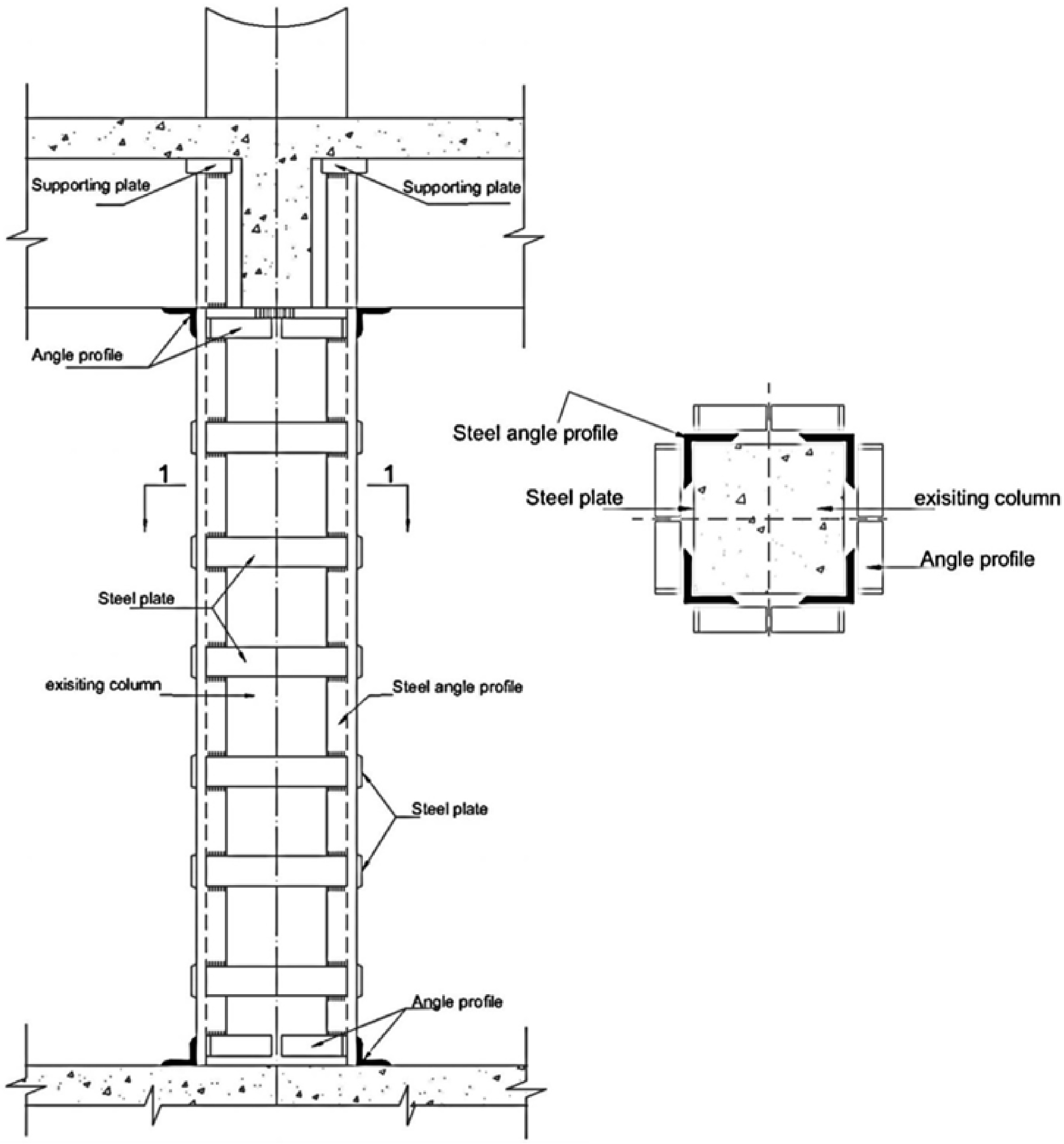


Figure 6.28 Steel jacketing of columns

6.2.3. ADDITION OF RC SHEAR WALL

The addition of new reinforced concrete shear walls provides the best option of strengthening an existing structure for improved seismic performance. It adds significant strength and stiffness to framed structures.

The minimum anchorage length of the grouted-in longitudinal and transverse reinforcement of the shear wall in to the existing components of the building shall not be less than 6 times the diameter of the bars.

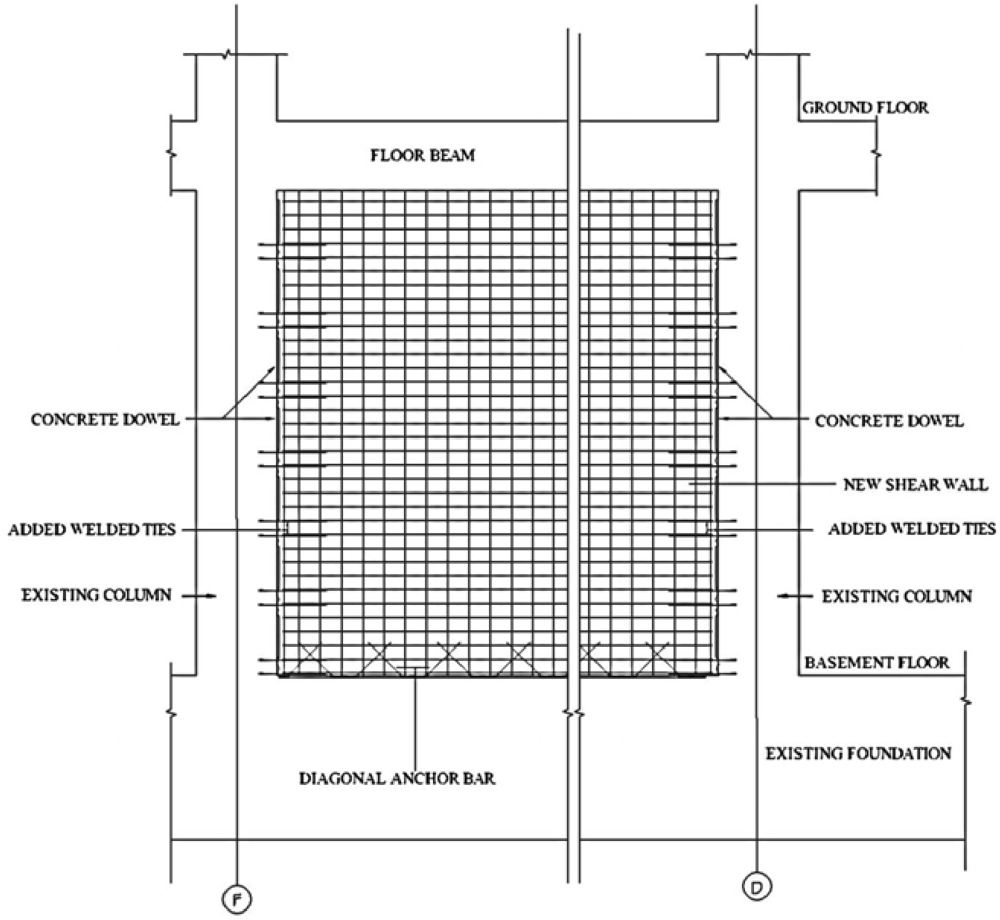


Figure 6.29 Addition of shear wall

Figure 6.30 Shear wall addition with column jacketing (source: MRB & Associates)

6.2.4. ADDITION OF STEEL BRACING

Steel diagonal braces can be added to the existing concrete frames. Braces should be arranged so that their center line passes through the centers of the beam – column joints. The brace connection should be adequate against out-of-plane failure and brittle fracture.

Figure 6.31 Addition of steel bracing

6.3. RECOMMENDED DETAILING FOR EARTHQUAKE RESISTANCE BUILDING

Figure 6.32 Beam column joint detailing (source: MRB & Associates)

Figure 6.33 Confining hoop made with single reinforcing bar (source: MRB & Associates)

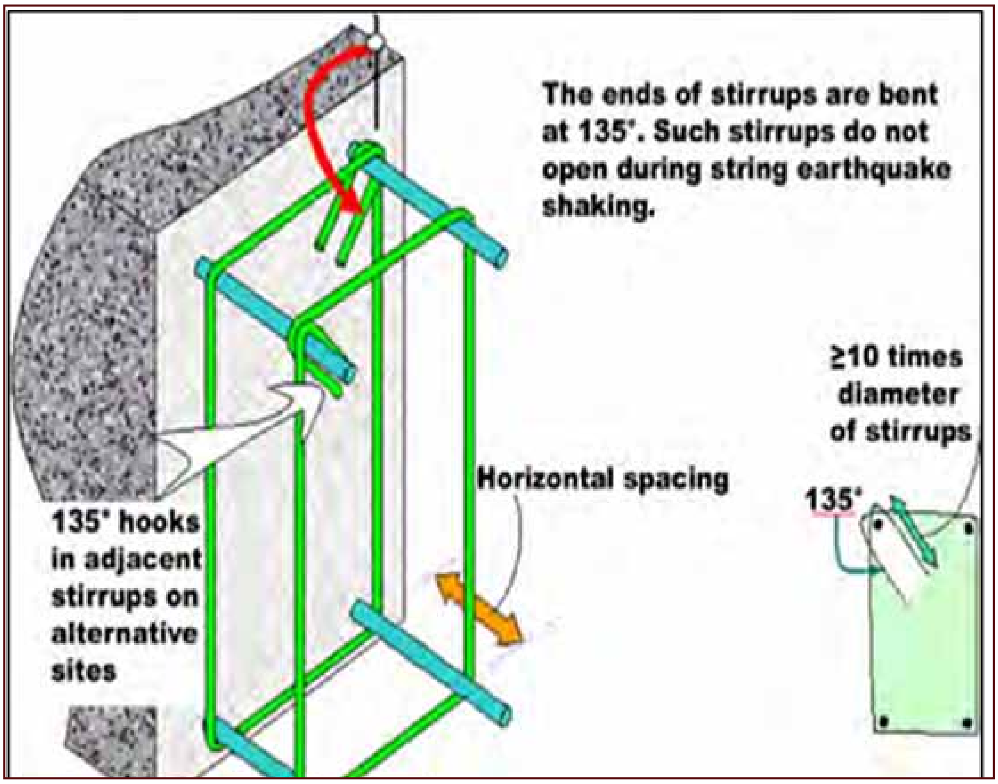


Figure 6.34 Stirrup detailing

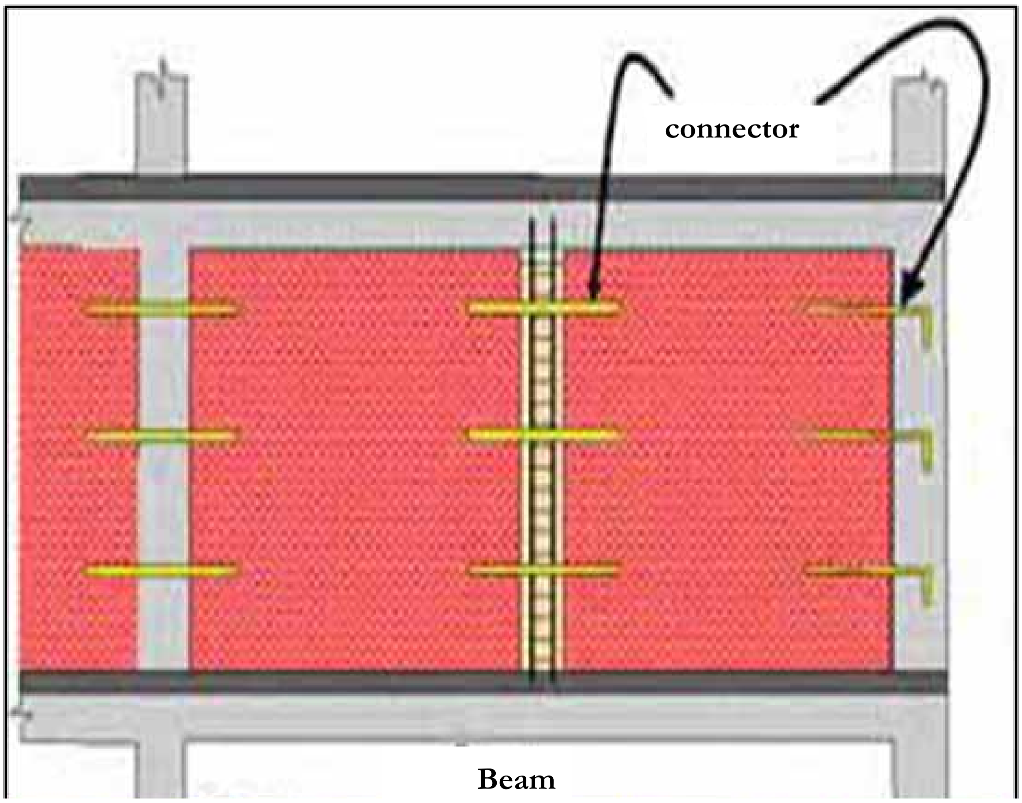


Figure 6.35 Detail of anchor between infill and the frame

7. STRUCTURAL VULNERABILITY ANALYSIS

Structural vulnerability analysis is very important to protect building or structures from damage. The structural vulnerability assessment is necessary due to many reasons. Structural vulnerability analysis may be necessary in Engineered or Non Engineered building; for many reasons, some of the reasons are listed below:

- Occupancy Change in the building
- Construction quality not appropriate
- Client interest
- Revision in the code
- Structural material degradation, etc.

1. Engineered Building:

In this category, buildings are designed with reference to codes and in the guidance of Engineer or Technical persons. But, vulnerability analysis or retrofitting may be required due to several reasons such as listed above.

2. Non Engineered Building:

In this category, buildings are built informally. These types of buildings are common in context of Nepal. These building are not structurally designed and supervised by engineers during construction. An example of structural vulnerability assessment and retrofit of engineered building has been demonstrated in this guideline in Annex A.

EXAMPLE 1: ENGINEERED RC FRAME BUILDING

A.1. ENGINEERED RC FRAME BUILDING

A.1.1. BUILDING DESCRIPTION

This building is RCC Frame structure in burnt clay bricks in cement mortar. The structure is 5-storey + 1-Basement with storey height of 4m and 3.8m. The floor consists of reinforced concrete slab system. The total height of the building is 25.28m. There are 230mm thick outer walls and light weight partition wall as inner walls.

This building is engineered building with sufficient column size and beam size.

Vulnerability analysis was done and retrofitting is recommended to correct L- shape by adding shear wall or retrofitting columns for torsion. Finally, comparisons of different retrofitting options are done to select the most appropriate retrofitting option.

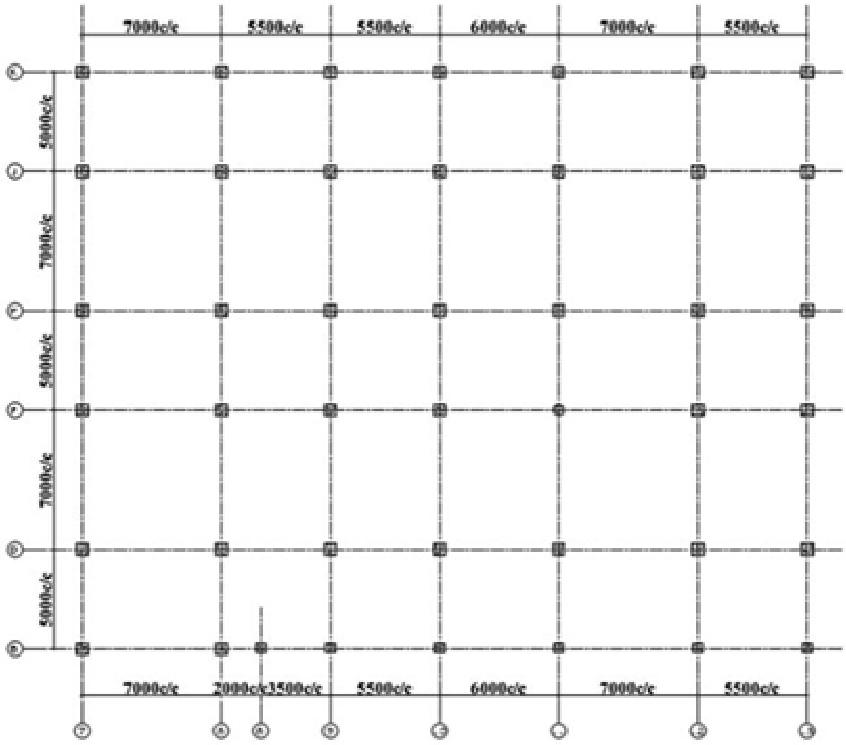
A.1.1.1. General Building Description

Building Plan Size	:	40.51m × 33m
No. of Story above ground level	:	5
No. of basement below ground level	:	1
Building Height	:	25.28m
Storey height	:	3.8m

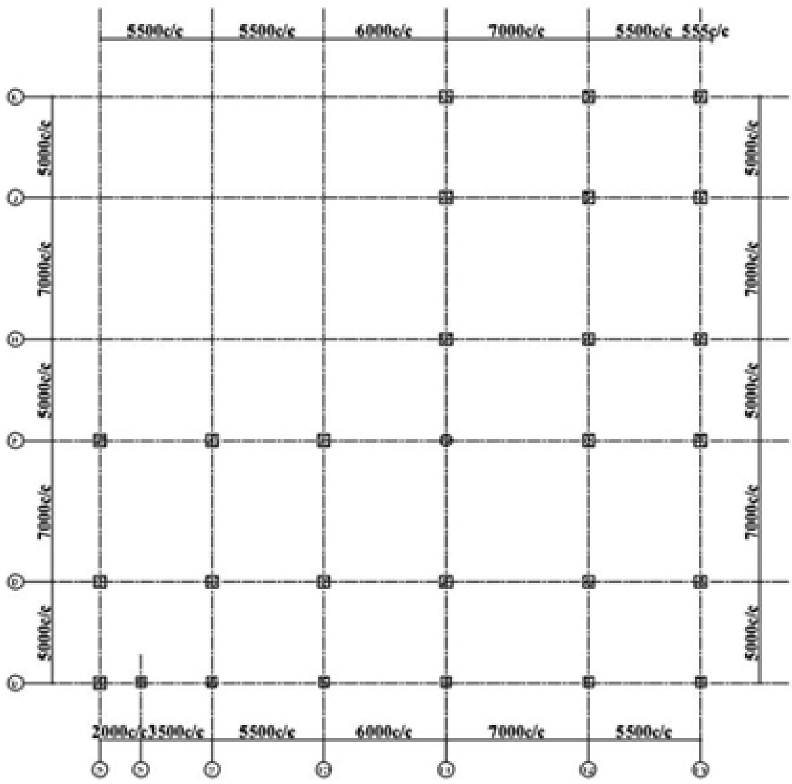
A.1.1.2. Structural System Description

Type of Structure	:	R.C Frame
Type of Foundation	:	Beam slab Footing
Roof Type	:	Sloped roof with clay tile
Column Sizes	:	400mm × 400mm , 500mm × 500mm, 600mm × 600mm, 700mm dia, 500mm dia, 600mm dia.
Beam Sizes	:	300mm × 550mm
Building Type	:	Building Type IV
Seismic Zone	:	1 (NBC 105:1994)

A.1.1.3. Building Drawings



BASEMENT PLAN



TYPICAL FLOOR PLAN

Figure A-1 Building drawings

A.1.2. ASSUMPTIONS

Unit weight of RCC= 25 KN/m^3
Unit weight of brick = 19.6 KN/m^3
Live load = 3.0 KN/m^2
Weight of plaster and floor finish = 0.73 KN/m^2 i.e. 22mm screed + 12mm plaster
Partition load = 1.2 KN/m^2
Grade of concrete = M20 for all the other structural elements
Grade of steel = Fe 415
Lateral load is solely carried by frame elements. Stiffness of the walls is not considered.

A.1.3. STRUCTURAL ASSESSMENT CHECKLIST

S.N.	CHECKS	REMARKS
1.	Load Path	The frame system provides a complete load path which transfers all inertial forces in the building to the foundation.
2.	Redundancy	There are more than two bays of frame in each direction.
3.	Geometry	The plan of the building is same in all stories except at basement. The building has basement for parking.
4.	Weak Storey / Soft Storey	There is no weak / soft storey.
5.	Vertical Discontinuities	Vertical elements in the lateral force resisting system are continuous to the foundation. Except for the basement columns.
6.	Mass	There is no change in effective mass in adjacent floors except at basement to ground floor.
7.	Torsion	The eccentricity of the building is not within the limit.
8.	Adjacent Buildings	There are no adjacent buildings.
9.	Short Column	No short column effect
10.	Deterioration of Concrete	No visible deterioration observed. No cracks were observed.

A.1.4. STRENGTH RELATED CHECKS

A.1.4.1. Calculation for Shear Stress Check

Lumped load

LEVEL	Combination DL+0.25LL	Seismic Weight (KN)	
6.00	6342.59	6342.59	
5.00	6073.94	6073.94	
4.00	6124.08	6124.08	
3.00	6132.29	6132.29	
2.00	6068.88	6068.88	
1.00	15717.80	15717.80	
	Σ	46459.57	KN

A.1.4.2. Calculation of Base Shear (Using NBC 105:1994)

Design Horizontal Seismic Coefficient for the Seismic Coefficient Method

The design horizontal seismic force coefficient, C_d shall be taken as :

$$C_d = C\ Z\ I\ K$$

Where, C is the basic seismic coefficient for the fundamental translational period in the direction under consideration.

Z = Seismic Zoning Factor I = Importance Factor

K = Structural Performance Factor

The total design lateral force or Design Seismic Base Shear (V_B) along any principal direction is determined by the following expression :

$$V_B = C_d W_t$$

Where, C_d = The Design Horizontal Seismic Coefficient

W_t = Total of the gravity loads of the whole building

The approximate fundamental natural period of vibration (T_a) in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

$$T_a = \frac{0.09h}{\sqrt{d}}$$

Where,

h = Height of Building in meter = 25.58m

d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force

dx = 40.51m

dz = 33m

$$T_{ax} = \frac{0.09h}{\sqrt{dx}} = 0.3617$$

$$T_{az} = \frac{0.09h}{\sqrt{dz}} = 0.4$$

Therefore ,C = 0.08 for medium soil

Seismic zoning factor for Kathmandu is, Z = 1.0

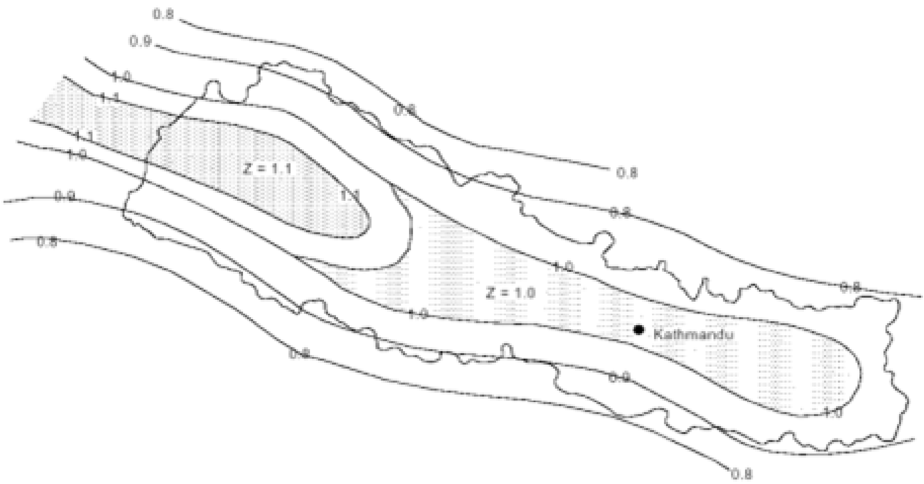


Figure A-2 Seismic zone for Kathmandu

C_d = C Z I K
= 0.08 ×1× 1×1
= 0.08

Base shear = $V_b = C_d W_t$
= 3716.766 KN

A.1.4.3. Distribution of Base Shear And Calculation Of Shear Stress In RC Columns

The horizontal seismic force at each level i shall be taken as :
The design base shear (V) computed in 1.5 shall be distributed along the height of the building as per the following expression:

$$\mathbf{F_i = V \times \frac{W_i h_i}{\sum W_i h_i}}$$

Where,

W_i = proportion of W_t contributed by level i,
h_i = Height of floor i measured from base

Floor	Total weight W_i (KN)	Height h_i (m)	W_i×h_i	W_i h_i ∑W_i h_i	Q_i(KN)	Storey Shear V_i (KN)
6	6342.59	23.000	145879.64	0.271	1007.521	1007.521
5	6073.94	19.200	116619.69	0.217	805.436	1812.957
4	6124.08	15.400	94310.77	0.175	651.359	2464.317
3	6132.29	11.600	71134.52	0.132	491.292	2955.609
2	6068.88	7.800	47337.23	0.088	326.936	3282.544
1	15717.80	4.000	62871.19	0.117	434.221	3716.766
∑			**538153.04**			

(Using IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of Buildings, 6.5.1)

Average Shearing stress in columns is given as

$T_{col} = \left(\frac{n_c}{n_c - n_f}\right) \times \left(\frac{v_j}{A_c}\right)$ < min. of 0.4 Mpa and $0.1\sqrt{f_{ck}}$ (Ref IS:15988:2013 Clause 6.5.1)

$0.1\sqrt{f_{ck}} = 0.1\sqrt{20} = 0.45$

For Ground Storey columns,

n_c = Total no. of Columns resisting lateral forces in the direction of loading
n_f = Total no. of frames in the direction of loading
A_c = Summation of the cross- section area of all columns in the storey under consideration
V_j = Maximum Storey shear at storey level 'j'
DCR = Demand Capacity Ratio

Storey	n_c	n_{f1}	n_{f2}	A_c	Storey Shears	Shear Stress (σ)		DCR		Remarks
						Colx (MPa)	Colz (MPa)	in x-dir	in z-dir	
6	28	6	6	8.737	1007.52	0.15	0.15	0.37	0.37	Since Demand Capacity Ratio is less than 1, hence Safe in shear
5	28	6	6	8.737	1812.96	0.26	0.26	0.66	0.66	
4	28	6	6	8.823	2464.32	0.36	0.36	0.89	0.89	
3	28	6	6	9.593	2955.61	0.39	0.39	0.98	0.98	
2	28	6	6	10.80	3282.54	0.39	0.39	0.97	0.97	
1	43	6	7	17.12	3716.77	0.25	0.26	0.63	0.65	

T_{col} < min of 0.4 MPa

Hence the check is satisfied

A.1.4.4. Axial Stress Check

Axial Stresses Due To Overturning Forces As Per FEMA310 (Clause 3.5.3.6)

i. Axial stress in moment frames for x-direction loading

Axial force in columns of moment frames at base due to overturning forces,
The axial stress of columns subjected to overturning forces F_o is given by

$$F_o = \frac{2}{3}\left(\frac{V_b}{n_f}\right) \times \left(\frac{H}{L}\right)$$

V_b = Base shear x Load Factor
3716.8 x1.5 = 5575.15 KN
A_c = column area = 17.12 Sq.m.
H =total height = 24 m
L =Length of the building = 40.51 m

$$F_o = \frac{2}{3}\left(\frac{V_b}{n_f}\right) \times \left(\frac{H}{L}\right)$$

= 275.25 KN

Axial Stress for x-direction loading,

$$\sigma = \frac{275.25 \text{ X } 1000}{0.16} = 1.72 \text{ MPa}$$

σ_{all} = 0.25 f_{ck}(Reference IS15988:2013 Clause 6.5.4) = 5.00 MPa
therefore $\sigma < \sigma_{all}$ OK
DCR = 0.334

ii. Axial stress in moment frames for z-direction loading

Axial force in columns of moment frames at base due to overturning forces,
The axial stress of columns subjected to overturning forces F_o is given by

$$F_o = \frac{2}{3}\left(\frac{V_b}{n_f}\right) \times \left(\frac{H}{L}\right)$$

V_b = Base shear x Load Factor   Load Factor = 1.5   (Ref: IS456:2000)
3716.8 x1.5 = 5575.15 KN

A_c = Column area = 17.12 sq.m.
H = Total height = 24 m
L = Length of the building = 33.00 m

$$F_o = \frac{2}{3}\left(\frac{V_b}{n_f}\right) \times \left(\frac{H}{L}\right)$$

= 337.89 KN

Axial Stress for z-direction loading,

$$\sigma = \frac{337.89 \times 1000}{0.16} = 2.11 \text{ MPa}$$

σ_{all} = 0.25 f_{ck} (Reference IS15988:2013 Clause 6.5.4) = 5.00 MPa

therefore $\sigma < \sigma_{all}$ OK

DCR = 0.422<1
Hence the check is satisfied

A.1.4.5. Check for Out-Of-Plane Stability of Brick Masonry Walls

Wall Type	Wall Thickness	Recommended Height/ Thickness ratio (0.24 <Sx ≤ 0.35) (Ref: Fema:310 Table 4-2)	Actual Height/ Thickness ratio in building	Comments
Wall in ground storey	230mm	18	$\frac{(3800 - 450)}{230} = 14.56$	Pass
Wall in upper stories	230mm	16	$\frac{(3800 - 450)}{230} = 14.56$	Pass

The out of plain stability of ground floor wall and that for the upper stories are within the permissible limit, hence the check is satisfied.

A.1.5. DETAILED ANALYSIS

A.1.5.1. Column Flexure Capacity

Calculating the column bending capacity for ground storey column:
The column demand given by load case with maximum value is:

P_u = 2572.5 KN
M_u = 517 KNm
f_{ck} = 20 Mpa
f_y = 415 Mpa
Clear cover = 40 mm

$d' = 40 + 10 + \frac{25}{2} = 62.5$

$\frac{d'}{D} = 0.104 \approx 0.1$

$A_s = 4555.278\ mm^2$

Percentage of reinforcement,

$p = 1.265\%$

$\frac{p}{f_{ck}} = 0.063$

$$\frac{P_u}{(f_{ck}bd)} = \frac{2572.51}{(20 \times 600 \times 600)}$$

$= 0.0378$

Referring to chart 44 of SP:16,

$\frac{M_u'}{(f_{ck}bD^2)} = 0.095$

M_u' = 410.4 KNm
DCR = 1.259>1

Hence the check is not satisfied.

A.1.5.2. Shear Capacity of Column

Considering that the steel in one face will be in tension,

$$A_s = 3 \times \pi \times \frac{25^2}{4}$$

$= 1472.62\ mm^2$

Therefore, 100As/bd =100×1472.62/600×537.5 = 0.456

For Pt = 0.456 and M20 grade of Concrete, From IS456:2000 Table19 τ_c = 0.47 Mpa
Stirrups are 4- legged, 10mm Ø @ 200mm c/c spacing

Then,

$= V_{us} = \frac{0.87 \times f_y \times A_{st} \times d}{S_v}$ Ref:IS:456:2000 Clause 40.4

$$= \frac{0.87 \times 415 \times 314.16 \times 537.5}{200}$$

= 304 KN

Therefore, V_u = $V_{us} + T_c bd$ (Ref:IS:456:2000 Clause 40.4)
= 456 KN

Shear force per analysis = 332 KN

Moment Capacity of Beam

$M_{u,lim}^{bR}$ = 0.138*fck*b*d²(Ref IS 456 Annex G)=194.27KNm

$M_{u,lim}^{bL}$ = 0.138fckbd²(Ref IS 456 Annex G)= 497.67 KNm

h_{st} = 3.8 m

V from capacity design (IS13920)

$$= V_u = 1.4 \times \frac{M_{u,lim}^{bL} + M_{u,lim}^{bR}}{h_{st}}$$

Hence, V_u = 254.925 KN

So, Final shear demand = 332 KN

V_u (=456 KN) > Shear demand

DCR = 0.728

Hence, the check is satisfied.

A.1.5.3. Shear Capacity of Beam

The shear reinforcement provided in the existing beam at support section is 2-legged 10Φ @ 100mm c/c.

As = 4-20Φ = 1257 mm²

$p_t = \frac{100A_s}{bd} = \frac{100 \times 1257}{300 \times 515} = 0.813\%$

Using table 19 of IS456:2000, for M20 grade of concrete and $\frac{100A_s}{bd}$ = 0.813,

τ_c = 0.575 MPa

Stirrups are 2-legged 10Φ @ 100mm c/c, hence from cl. 40.4 of IS456:2000

$$V_{us} = \frac{0.87 \times f_y \times A_{st} \times d}{S_v}$$

$V_u = V_{us} + \tau_c bd$

$$= \frac{0.87 \times 415 \times 2 \times 78.57 \times 515}{100} + 0.575 \times 300 \times 515 = 381.0 \text{ KN}$$

Shear Demand in beam:

V as per analysis = 293.9kN

Moment capacity of beam

M_R^H = 194.27 kNm

M_R^S = 497.67 kNm

L_c = 7-0.6 = 6.4 m

$V_a^{D+L} = V_b^{D+L}$ = 126 kN

V from capacity design (IS13920)

$= Vu = 126 + 1.4 \times \frac{(M_R^H + M_R^S)}{L_c}$

= 277.36 kN

Hence final shear demand in beam = 293.9kN

V_u (=381KN)> 293.9 KN

DCR = 0.771<1

Hence, the check is satisfied.

A.1.5.4. Check for Strong Column Weak Beam

The flexure strengths of the columns shall satisfy the condition:

From IS15988:2013 7.4.1 , ∑Mc ≥ 1.1 ∑Mb

- **Checking Capacity of Center Column at Ground Floor:**

The longitudinal beam of size 300×550 is reinforced with 3-20dia. + 3-25dia. (i.e 2415.09mm^2) at top and 4-20 dia. (ie 1256.636mm^2) at bottom.

Where,

b = 300mm; d= 515mm

The hogging and sagging moment capacities are evaluated as 303.406 KNm and 194.27 KNm respectively.

Factored column axial load = 4770 KN(1.2DL + 1.2Eqz + 1.2LL)

$\frac{P_u}{(f_{ck}bd)}$ = 0.6625 where column size is 600 mm × 600 mm

The column is reinforced with 8-25dia. + 2-20dia.

A_{sc} = 4555.278mm^2; p_t = 1.265% Referring to chart SP16 Clause 3.0 Chart

Therefore,

$\frac{M_u'}{(f_{ck}bD^2)}$ = 0.01

M_u = 43.2kNm

∑Mc = 43.2+ 43.2 = 86.4KNm

∑Mb = 303.406 + 194.27 = 497.676KNm

1.1∑Mb = 547.437KNm

∑Mc<<< 1.1 ∑Mb

Hence, check is not satisfied.

- **Checking Capacity of Center Column of Peripheral Frame at Ground Floor:**
The longitudinal beam of size 300 × 550 is reinforced with 3-20dia. + 2-25dia. (ie 1923.778mm^2) at top and 3-20 dia. (ie 942.477mm^2) at bottom.

Where,
b = 300mm; d= 515mm

The hogging and sagging moment capacities are evaluated as 265.3 KNm and 153.1 KNm respectively.

Factored column axial load = 2906.68 KN

$\frac{P_u}{(f_{ck}bd)}$ = 0.404 where column size is 600 mm × 600 mm

The column is reinforced with 8-25dia.

A_{sc} = 3928.56mm^2 ;pt = 1.09% from SP-16

Therefore,

$\frac{M'_u}{(f_{ck}bD^2)}$ = 0.065

M_u = 280.8 KNm

∑Mc = 280.8+280.8 = 561.6 KNm

∑Mb = 265.3 + 153.1 = 418.4 KNm

1.1∑Mb = 460.24 KNm
From IS15988:2013 7.4.1∑Mc> 1.1 ∑Mb

Hence, check is satisfied.

A.1.6. EVALUATION SUMMARY

- The building is safe in strength related checks such as shear stress capacity, axial stress, out of plane stability.
- The computer analysis of the structure shows:
  - Foundation: Safe
  - Beam : Safe
  - Column : Not Safe
    (The DCR lies in the range of 1.5 indicating more detailed analysis)
  - Floor slab: Safe
- Thus, the above evaluations state that the frame has to be strengthened and retrofitted.

A.1.7. RETROFITTING OPTIONS

A.1.7.1. Option1: RC Jacketing On Columns

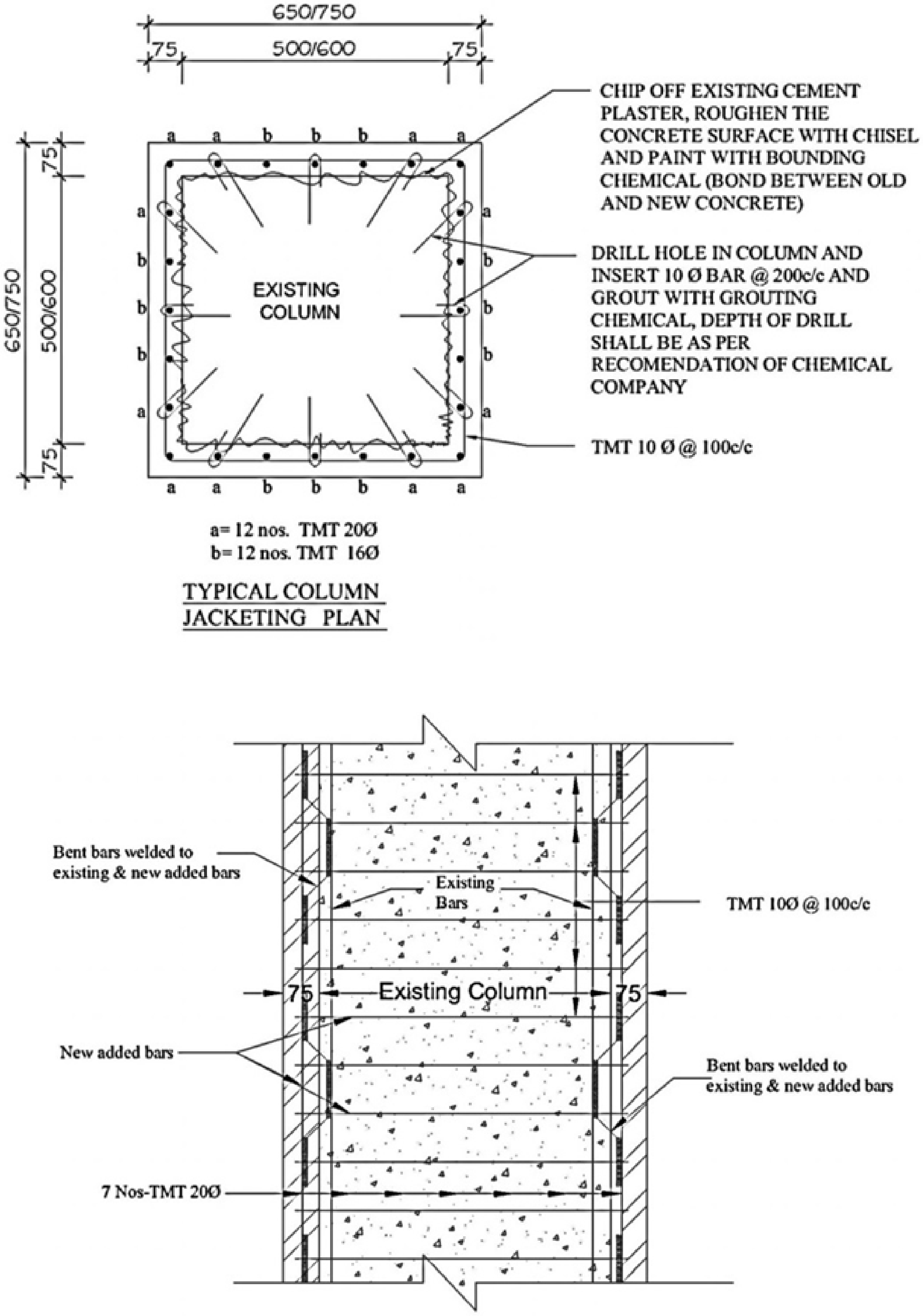


Figure A-3 Column Jacketing section

A.1.7.2. Option2: Steel Jacketing

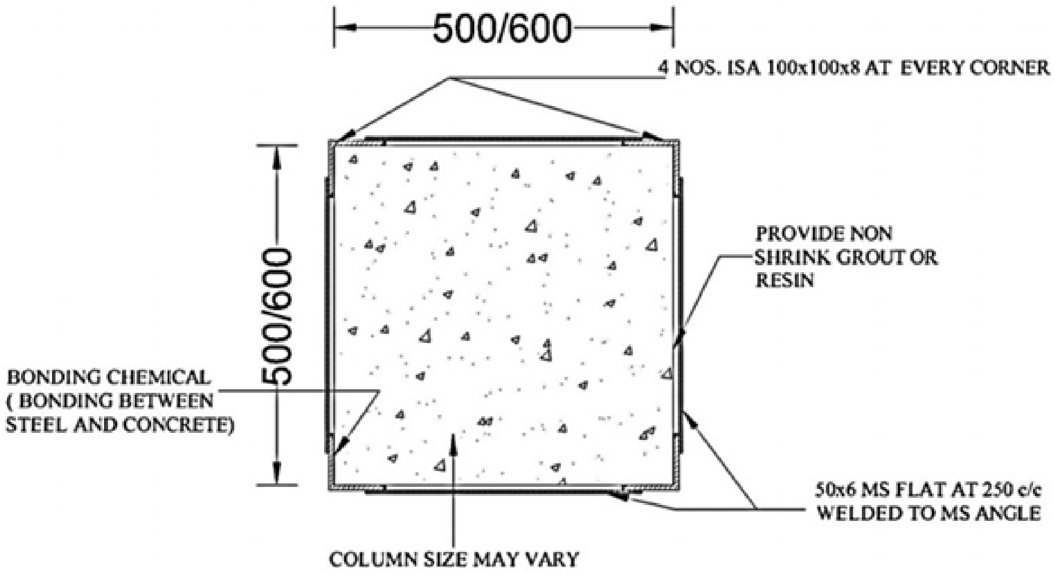


Figure A-4 Typical column steel jacketing detail plan

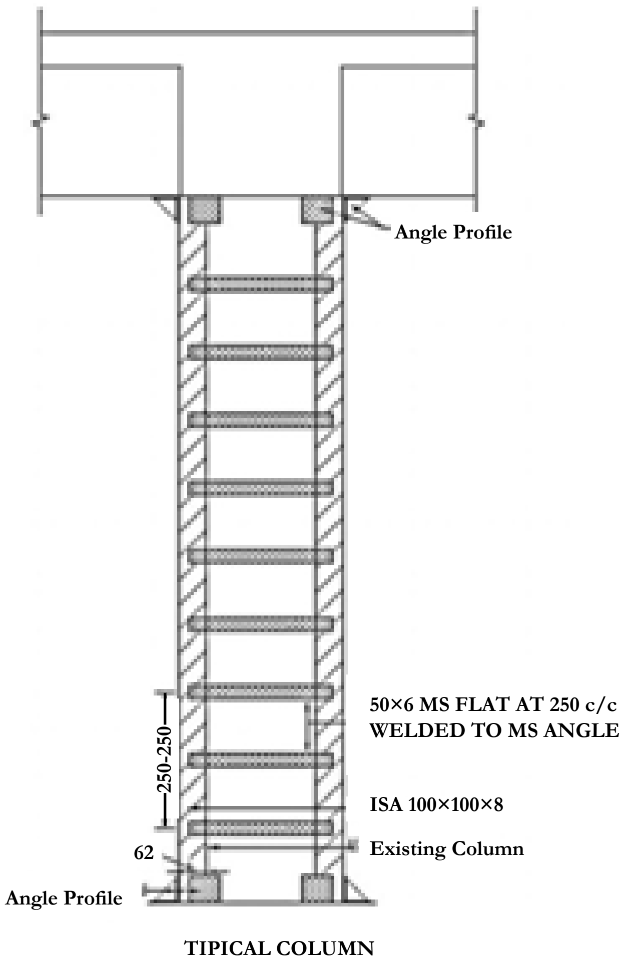


Figure A-5 Steel jacketing detail elevation

A.1.7.3. Option3: Shear Wall Addition with Column Jacketing

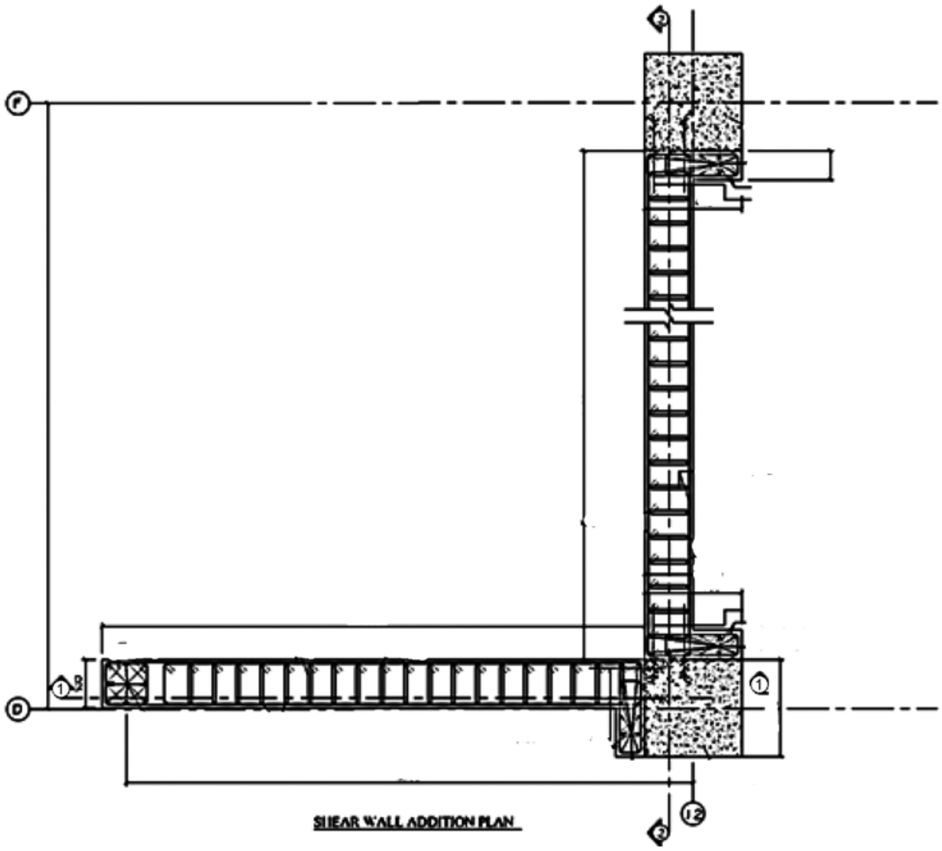
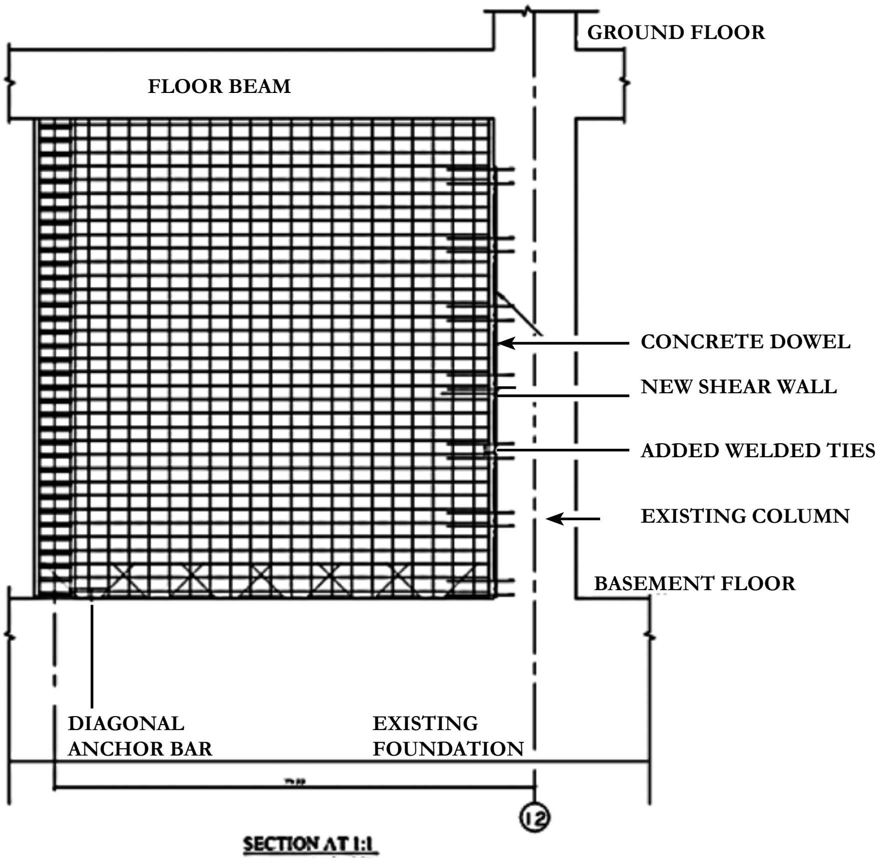


Figure A-6 Steel wall addition plan



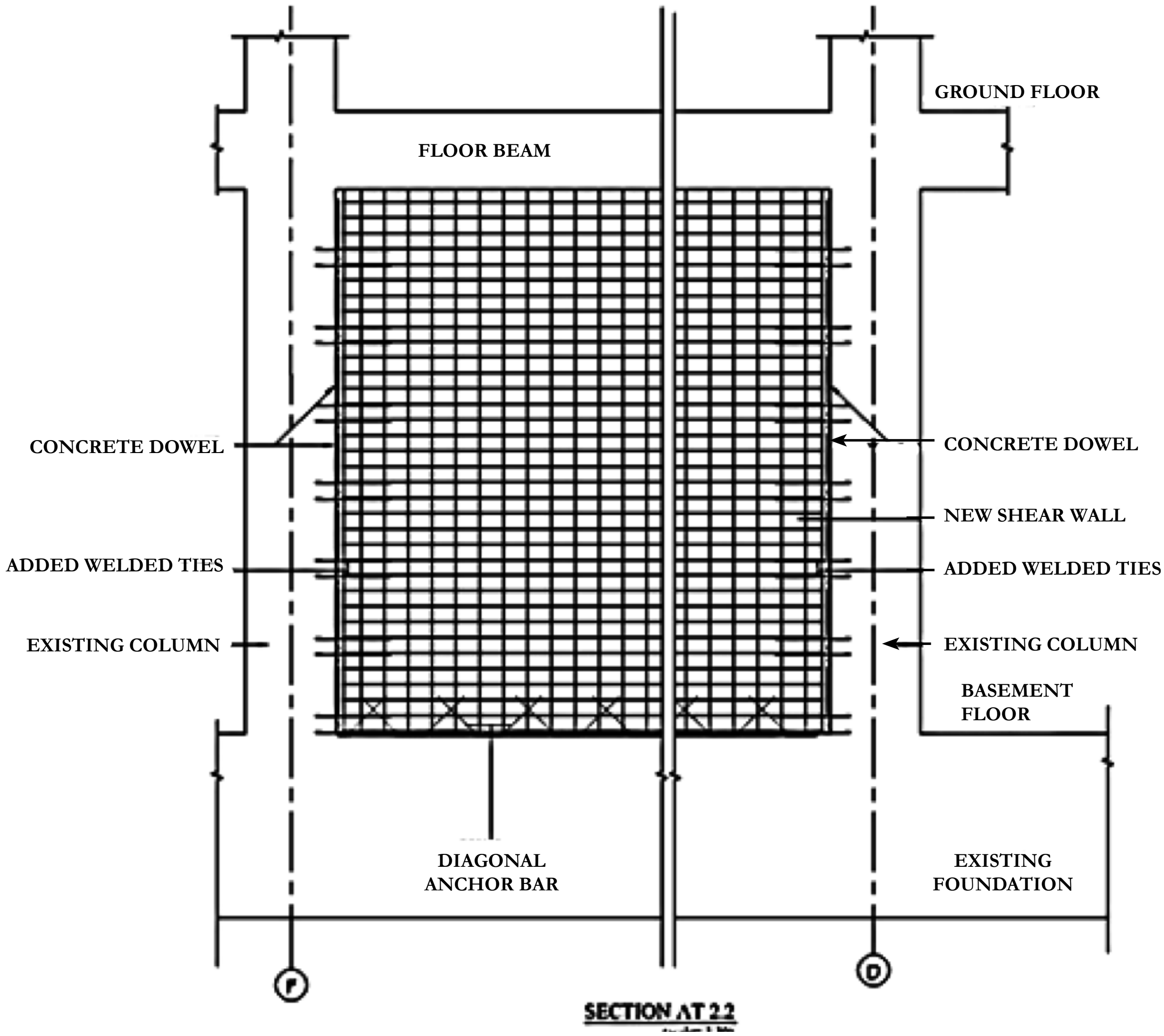


Figure A-7 Sections

A.1.8. COST ESTIMATION OF RETROFITTING OPTIONS

As per the District rate of 2012 AD, cost of the different options of building is mention below:

- Reinforced Concrete Jacketing on columns with approximate cost of NRs. 12,094,773
- Steel Jacketing on columns with approximate cost of NRs. 8,614,768
- Shear wall Addition and Column Jacketing with approximate cost of NRs. 8,176,350

S.N.	Alternatives	Disturbance to existing tenants	Estimated Time for work
1	RC Jacketing on column	High	6 months
2	Steel Jacketing on column	High	5 months
3	Shear wall addition and column jacketing	Medium	3.5 months

COMPARATIVE STUDY OF DIFFERENT OPTIONS

Alternatives	Options	Parameter						
		Disturbance to exiting structure	Time Consumption	Disturbance to existing function during constrction	Cost	Damagibility after retrofitting	Effection present aesthetic	Requirement of foundation strengthening
1	RC Jacketing on Column	***	***	***	***	*	**	*
2	Steel Jacketing on Column	***	**	***	**	**	**	*
3	Shear wall addition and column jacketing	**	*	**	**	**	*	**

**** High*
*** Medium*
** Low*

A.1.9. RECOMMENDATION

From the point of cost estimation and time of completion for the retrofitting, it is likely to adopt option 3, i.e. shear wall addition with concrete jacketing of columns.

EXAMPLE 2: OCCUPANCY CHANGE

A.2. SEISMIC EVALUATION OF RESIDENTIAL RCC BUILDING WHICH CONVERTED TO HEALTH CLINIC (OCCUPANCY CHANGE)

This building is RCC frame structure situated at Khusibu, Naya Bazar. This building is in good condition and well maintained but built before seismic code was introduced in Nepal.

The size of column is 230mm × 230mm, beam size of 230mm × 350mm, slab thickness of 125mm and storey height of 2.7m. It consists of 3- storey.

The building was built for the purpose of residential use. After the fast urbanization this locality of the building, Khusibu, is more commercial so now this building is to be converted into the health clinic.

A.2.1. GENERAL DESCRIPTION OF EXISTING BUILDING

Building Description : RCC Frame Structural (In good Condition, but built before Seismic Code introduced in NEPAL)	Site Visit/ Visual Inspection/Site measurements
Location : Khusibhu, Naya Bazar	Site Visit/ Visual Inspection/Site measurements
Storey height : 2.7 m	Site Visit/ Visual Inspection/Site measurements
No. of Stories : 3 nos	Site Visit/ Visual Inspection/Site measurements
Column Size : 230 mm × 230 mm	Site Visit/ Visual Inspection/Site measurements
Beam Size : 230 mm × 350 mm	Site Visit/ Visual Inspection/Site measurements
Slab thickness : 125 mm	Site Visit/ Visual Inspection/Site measurements
Type of foundation : Isolated foundation	Site Visit/ Foundation Exploration

A.2.2. STRUCTURAL ASSESSMENT CHECKLIST

S.N.	CHECKS	REMARKS
1.	Load Path	The frame system provides a complete load path which transfers all inertial forces in the building to the foundation.
2.	Redundancy	There are two bays of frame in each direction.
3.	Geometry	The plan of the building is same in all stories.
4.	Weak Storey / Soft Storey	There is no weak / soft storey.
5.	Vertical Discontinuities	Vertical elements in the lateral force resisting system are continuous to the foundation.
6.	Mass	There is no change in effective mass in adjacent floors except at top floor.
7.	Torsion	The eccentricity of the building is not within the limit.
8.	Adjacent Buildings	There are no adjacent buildings.
9.	Short Column	No short column effect
10.	Deterioration of Concrete	No visible deterioration observed. No cracks were observed.

A.2.3. BUILDING DRAWINGS

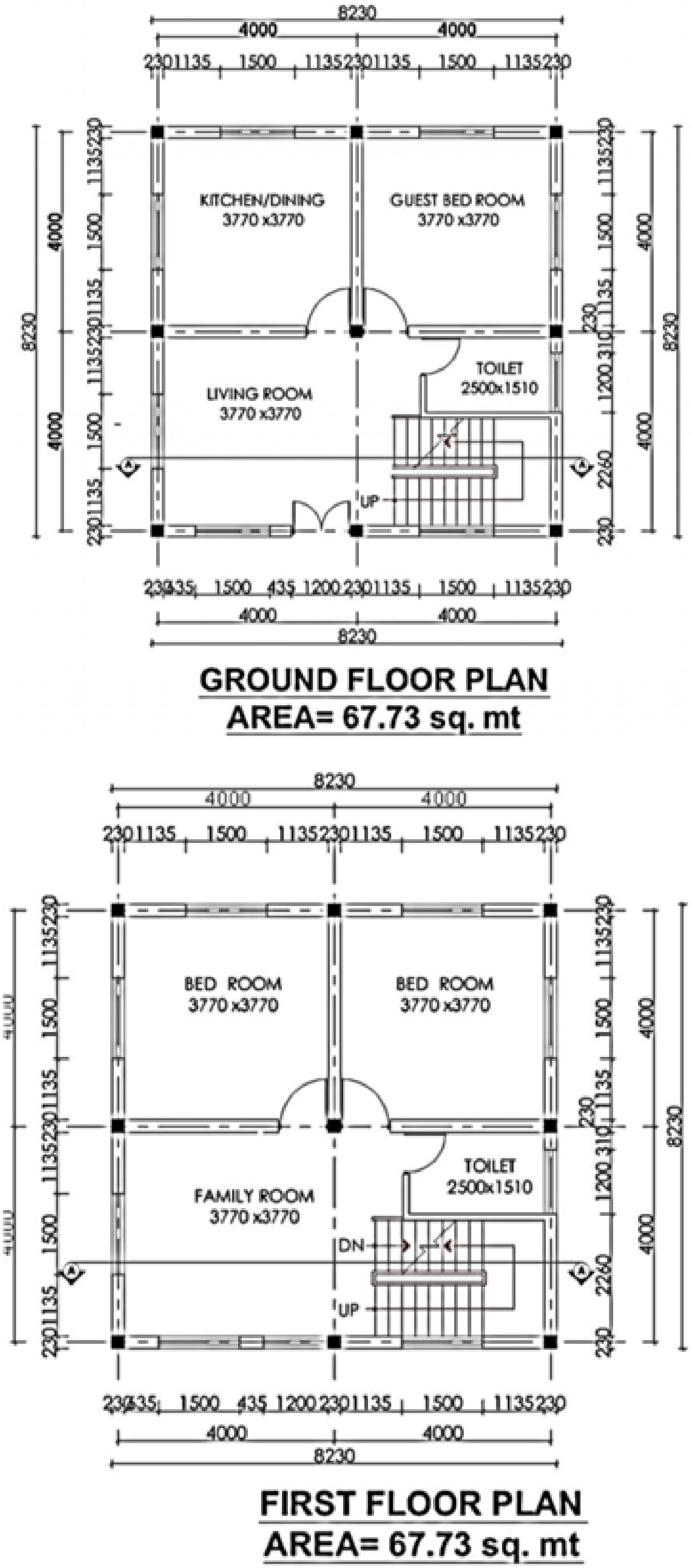


Figure A-8 Building plan

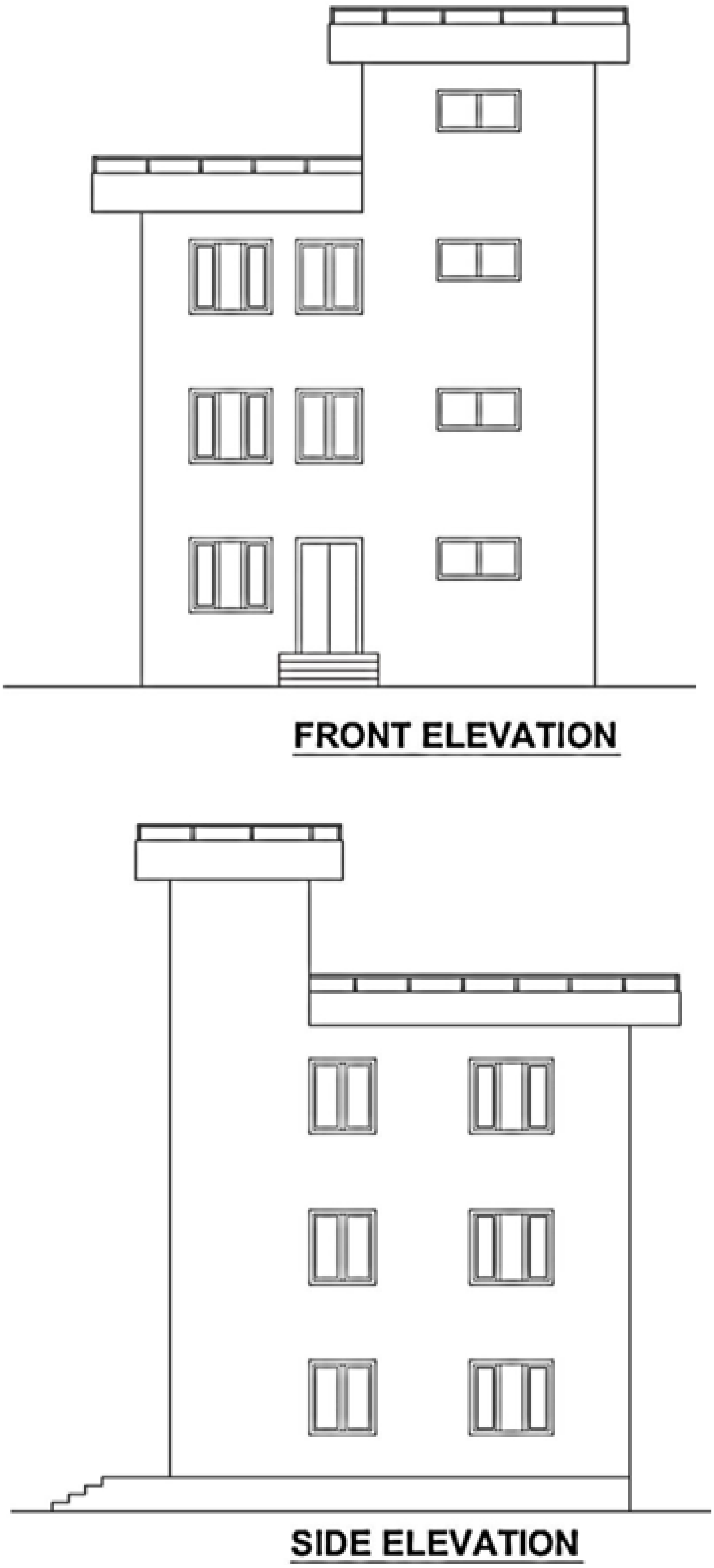


Figure A-9 Front and side elevation

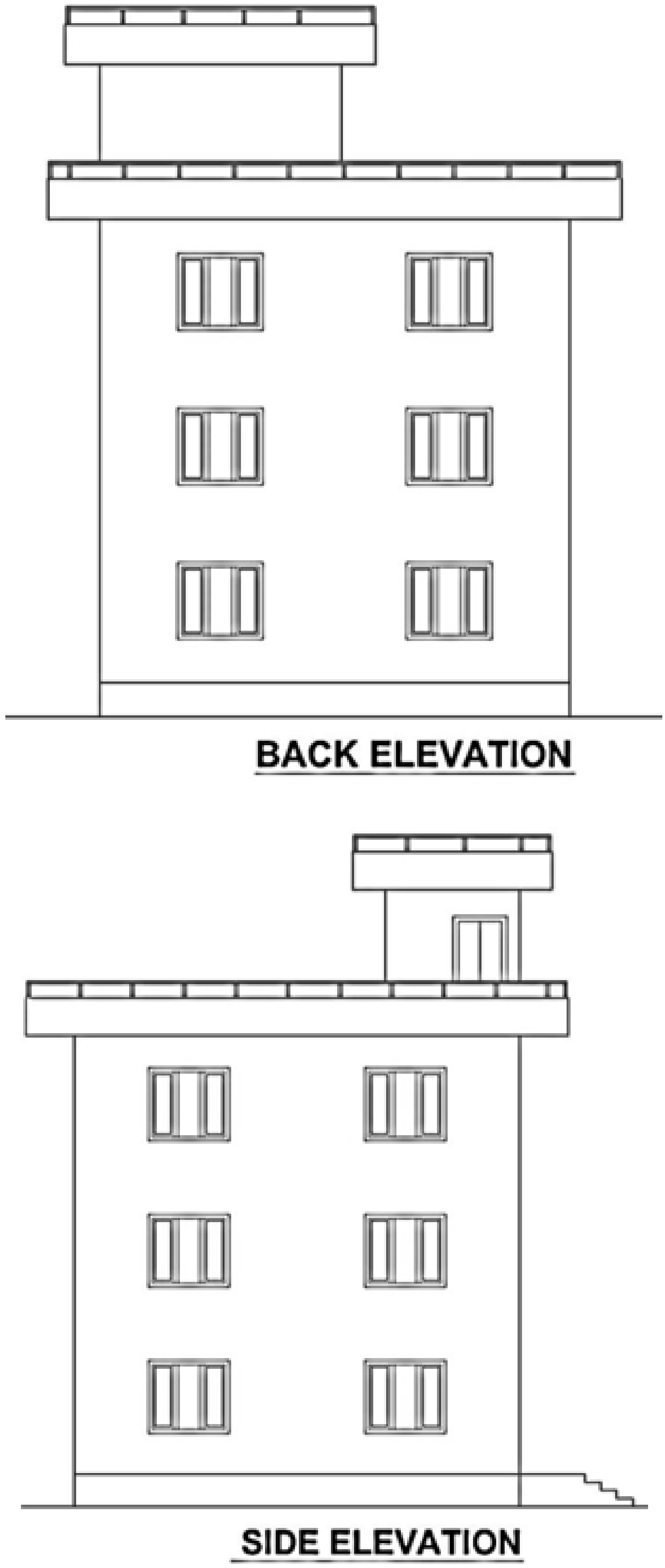


Figure A-10 Back and side elevation

A.2.4. STRUCTURAL DATA

Unit Weight of RCC = 25 KN/m^3
Unit Weight of Brick Masonry = 19.6 KN/m^3
Unit Weight of Plaster = 20 KN/m^3
Unit Weight of Marble = 26.7 KN/m^3

Live load:
For Floors = 2.5 KN/m^2 (Residential building)
= 3.0 KN/m^2 (Health Clinic)
For Roof = 1.5 KN/m^2
Grade of Concrete = M20 Site Visit/ Visual Inspection/Site Measurements
Grade of Steel = Fe 415 Site Visit/ Visual Inspection/Site Measurements
(Stiffness of the Brick Masonry is not considered in the calculation)

A.2.5. LOAD CALCULATIONS

Dead Load :

1. For Different Floors:

Slab Load = $0.125 \times 25 = 3.125$ KN/m^2
Ceiling Plaster Load = $0.02 \times 20 = 0.40$ KN/m^2
Floor Finish Load = $0.025 \times 20 = 0.50$ KN/m^2
Marble Floor Load = $0.025 \times 26.7 = 0.667$ KN/m^2
Total Load = 4.692 KN/m^2
~ 4.70 KN/m^2

2. For Roof Floor:

Slab Load = $0.125 \times 25 = 3.125$ KN/ m^2
Ceiling Plaster Load = $0.02 \times 20 = 0.40$ KN/ m^2
Floor Finish Load = $0.025 \times 20 = 0.50$ KN/ m^2
Mosaic Floor Load = $0.025 \times 20 = 0.50$ KN/ m^2
Total Load = 4.525 KN/m^2
~ 4.50 KN/m^2

A.2.6. Case I: STRUCTURAL ANALYSIS OF BUILDING (As Residential Building)

A.2.6.1. Method I: As per NBC 105:1994

Assumptions:

a) Live Load Calculation:

Unit weight of brick work = 19.6 KN/m^3
Live load = 2.5 KN/m^2

LEVEL	FLOORS	FLOOR AREA sq.m	LL	0.25LL	Remarks
4	Third Floor	29.485	44.2275	11.056875	
3	Second Floor	90.33	270.99	67.7475	
2	First Floor	67.73	203.19	50.7975	
1	Ground Floor	67.73	203.19	50.7975	
$\sum$				180.39938	

Note: Quick check calculation based on FEMA 310 for the seismic evaluation of building under consideration

b) Lump Mass Calculation:

S.NO.	FLOORS	Total Dead Load (KN)	Total Live Load (KN)	Total Weight (KN)	Remarks
4	Third Floor	260.48	11.056875	271.54	
3	Second Floor	756.04	67.7475	823.79	
2	FirstFloor	649.82	50.7975	700.62	
1	Ground Floor	649.82	50.7975	700.62	
$\sum$				2496.56	

c) Calculation of Base Shear

The design horizontal seismic force coefficient, C_d shall be taken as :

As per NBC 105 $\quad C_d = C\ Z\ I\ K$

Where, C is the basic seismic coefficient for the fundamental translational period in the direction under consideration.

Z = Seismic Zoning Factor
I = Importance Factor
K = Structural Performance Factor

The total design lateral force or Design Seismic Base Shear (V_B) along any principal direction is determined by the following expression :

$$V_B = C_d W_t$$

Where, C_d = The Design Horizontal Seismic Coefficient
W_t = Total of the gravity loads of the whole building

The approximate fundamental natural period of vibration (T_a) in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

As per NBC 105 $\quad T_a = \frac{0.09h}{\sqrt{d}}$

Where, h = Height of Building in meter = 10.8m
d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force

dx = 8.23m
dz = 8.23m

$T_{ax} = \frac{0.09h}{\sqrt{dx}}$
$= 0.338$

$T_{az} = \frac{0.09h}{\sqrt{dz}}$
$= 0.338$

Therefore, C = 0.08 for medium soil (Ref: NBC 105:1994)

Seismic zoning factor for Lalitpur is, Z = 1.0

C_d = C Z I K
= 0.08 X 1 X 1 X 1
= 0.08

Base shear **= V= C_dW_t**
= 0.08 × 2468.34 197.46KN

d) Distribution of Base Shear And Calculation Of Shear Stress In RC Columns

The horizontal seismic force at each level i shall be taken as :

The design base shear (V) computed shall be distributed along the height of the building as per the following expression:

As per NBC105 $F_i = V \times \frac{W_i h_i}{\sum W_i h_i}$

Where,

W_i = Proportion of W_t contributed by level i,
h_i = Height of floor i measured from base

Floor	Total weight Wi (KN	Height h_i (m)	$W_i h_i$	$\frac{W_i h_i}{\sum W_i h_i}$	Qi (KN) $= V \times \frac{W_i h_i}{\sum W_i h_i}$	Storey Shear F_i (KN)
4.00	271.54	10.8	2932.63	0.19	37.51	37.51
3.00	812.50	8.1	6581.25	0.43	84.90	122.41
2.00	692.15	5.4	3737.61	0.24	47.39	169.8
1.00	692.15	2.7	1868.80	0.12	23.69	193.49
∑	2468.34		15120.29	1.00	370.25	

(Using IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of Buildings, 6.5.1)

Average Shearing stress in columns is given as

$T_{col} = \frac{n_c}{n_c - n_f} \times \frac{V_j}{A_c} <$ min of 0.4MPa and $0.1\sqrt{f_{ck}}$ (Ref: IS 15988:2013 Clause 6.5.1)

$0.1\sqrt{fck}$ = 0.45

For Ground Storey columns,

n_c = Total no. of Columns resisting lateral forces in the direction of loading
n_f = Total no. of frames in the direction of loading
A_c = Summation of the cross- section area of all columns in the storey under consideration
V_j = Maximum Storey shear at storey level 'j'
DCR = Demand Capacity Ratio

Storey	n_c	n_{f1}	n_{f2}	A_c	Storey Shears	Shear Stress		DCR		Remarks
						T col-x (MPa)	T col-z (MPa)	in x- dir	in z-dir	
4	4	2	2	0.211	37.51	0.35	0.35	0.87	0.87	Demand Capacity Ratio is not less than 1,
3	9	3	3	0.476	122.41	0.38	0.38	0.95	0.95	
2	9	3	3	0.476	169.8	0.53	0.53	1.32	1.32	
1	9	3	3	0.476	193.49	0.6	0.6	1.5	1.5	

T_{col}> min of 0.4 MPa and $0.1\sqrt{f_{ck}}$ = 0.45MPa

Hence the check is not satisfied

e) Axial Stress Check

Axial Stresses Due To Overturning Forces As Per FEMA 310

i) Axial stress in moment frames for x-direction loading

Axial force in columns of moment frames at base due to overturning forces,
The axial stress of columns subjected to overturning forces Fo is given by

$F_o = \frac{2}{3}\frac{V_b}{n_f} \times \frac{H}{L}$ Fema 310 Clause 3.5.3.6

V_b = Base shear x Load Factor

370.25 x1.5	=	555.375	KN
A_c = column area	=	0.0529	Sq.m.
H= total height	=	10.8	m
L=Length of the building	=	8.00	m

$= \frac{2}{3}\frac{V_b}{n_f} \times \frac{H}{L}$

= 166.61 KN

Axial Stress for x-direction loading,

$\sigma = \frac{166.61 \times 1000}{0.05} = 3.33$ MPa

$\sigma_{all} = 0.25\, f_{ck}$
= 5.00 MPa (IS 15988:2013Clause 6.5.4)

therefore $\sigma < \sigma_{all}$ OK

ii) Axial stress in moment frames for z-direction loading

Axial force in columns of moment frames at base due to overturning forces,

The axial stress of columns subjected to overturning forces Fo is given by

$F_o = \frac{2}{3}\frac{V_b}{n_f} \times \frac{H}{L}$

V_b = Base shear x Load Factor

370.25 × 1.5 = 555.375 KN

A_c = Column area = 0.052 sq.m.

H = Total height = 10.8 m

L = Tength of the building = 8.00 m

$F_o = \frac{2}{3}\frac{V_b}{n_f} \times \frac{H}{L}$ Fema 310 Clause 3.5.3.6

= 166.61 KN

Axial Stress for z-direction loading,

$\sigma = \frac{166.61 \times 1000}{0.052} = 2.11$ MPa

(IS 15988:2013Clause 6.5.4)σ_{all} = 0.25 fck = 5.00 MPa

therefore $\sigma < \sigma_{all}$ OK

Hence the check is satisfied

f) Checking shear capacity of beam

The shear reinforcement provided in the existing beam at support section is 3 TOR 16 Top and bottom

Where,

b = 230

D = 350

d = 350 – 25 – 8 = 317

Area of steel (A_{st}) = 4 tor 12 diameter = 452mm²

f_{ck} = 20N/mm²

f_y = 415N/mm²

$P_t = \frac{100A_{st}}{bd} = \frac{100 \times 452}{230 \times 317} = 0.619$ %

Using table 19 of IS 456: 2000, for M20 grade of concrete and $\frac{100A_{st}}{bd}$ = 0.813

τ_c = 0.58 MPa

Stirrups are 2- legged 8 mm dia. @ 150mm C/C, hence from clause 40.4 of IS 456:2000

$V_{us} = \frac{0.87 \times f_y \times A_{st} \times d}{S_v}$

$V_u = V_{us} + \tau_c bd$

$= \frac{0.87 \times 415 \times 2 \times 50.265 \times 317}{150} + 0.58 \times 230 \times 317$

= 118.99KN

A.2.6.2. Method II: As per IS 1893:2002-Part 1

Assumptions:

Unit weight of brick work = 19.6 KN/m^3

Live load = 2.5 KN/m^2

a) LIVE LOAD CALCULATION

LEVEL	FLOORS	FLOOR AREA (Sq.m)	LL (KN/m^2)	0.25LL	Remarks
4	Third Floor	29.485	44.2275	11.057	
3	Second Floor	90.33	225.825	56.456	
2	First Floor	67.73	169.325	42.331	
1	Ground Floor	67.73	169.325	42.331	
∑				152.18	

Note: Quick check calculation based on FEMA 310 for the seismic evaluation of building under consideration.

b) LUMP MASS CALCULATION

S.NO.	FLOORS	Total Dead Load (KN)	Total Live Load (KN)	Total Weight (KN)	Remarks
4	Third Floor	260.48	11.056875	271.54	
3	Second Floor	756.04	56.45625	812.50	
2	First Floor	649.82	42.33125	692.15	
1	Ground Floor	649.82	42.33125	692.15	
∑				2468.34	

c) CALCULATION OF BASE SHEAR

Calculation of earthquake loads using Seismic coefficient method:

The design horizontal seismic coefficient,

$$A_h = \frac{ZIS_a}{2Rg}$$

Reference IS:1893:2002

Where Z = Zone Factor

I = Importance Factor

R = Response Reduction Factor

$\frac{S_a}{g}$ = Average Response Acceleration Coefficient

The total design lateral force or Design Seismic Base Shear (V_B) along any principal direction is determined by the following expression :

$V_B = A_h W$

Where,

A_h = The Design Horizontal Seismic Coefficient

W = Seismic weight of the building

The approximate fundamental natural period of vibration (T_a) in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

$$T_a = \frac{0.09h}{\sqrt{d}}$$

Ref : IS1893:2002

Where,

h = Height of Building in meter = 10.80 m

d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force

dx = 8.23 m

dz = 8.23 m

$T_{ax} = \frac{0.09h}{dx^{0.5}}$

= 0.338 <0.55

$T_{az} = \frac{0.09h}{dz^{0.5}}$

= 0.338 <0.55

Therefore ,

$\frac{S_a}{g}$ = 2.5 for medium soil (IS :1893(Part 1) : 2002

Z = 0.36 (For Seismic Zone V) (Refer IS 1893 (Part 1) :2002-table 2)

I = 1.0 (For Residential Building) (Refer IS 1893 (Part 1) :2002-table 6)

$\frac{S_a}{g}$ = 2.5 (For Medium Soil) (Refer IS 1893 (Part 1):2002-Clause 6.4.5 and Fig.2)

R = 3.0 (For Ordinary RC Moment Resisting Frame) (Refer IS 1893 (Part 1) :2002-table 7)

The total design lateral force or design seismic base shear is given by,

$$A_h = \frac{ZIS_a}{2Rg}$$

= 0.36 * 1.0 *2.5/ 2 * 3 = 0.15

Base shear = $V_b = A_h W$

= 0.15 $\times$ 2468.34

= 370.251 KN

d) Distribution of Base Shear and Calculation of Shear Stress in RC Columns :

Floor	Total weight W_i (KN)	Height h_i (m)	$W_i h_i^2$	$\frac{W_i h_i^2}{\sum W_i h_i^2}$	Q_i(KN)	Storey Shear V_i (KN)
4.00	271.54	10.8	31672.06	0.29	106.40	106.40
3.00	812.50	8.1	53307.88	0.48	179.09	285.49
2.00	692.15	5.4	20183.13	0.18	67.81	353.30
1.00	692.15	2.7	5045.78	0.05	16.95	370.25
$\sum$	2468.34		110208.85	1.00	370.25	

e) SHEAR STRESS AT STOREY LEVEL :

(Using IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of buildings 6.5.1)

The Total design lateral force or design seismic base shear is given by,

$$V_b = A_h W$$

Average Shearing stress in columns is given as (Ref: IS 15988:2013 Clause 6.5.1)

$$T_{col} = \left(\frac{n_c}{n_c - n_f}\right) \times \left(\frac{v_j}{A_c}\right) < \text{min. of } 0.4 \text{ Mpa and } 0.1\sqrt{f_{ck}}$$

For Ground Storey columns,

- n_c = Total No. of Columns resisting lateral forces in the direction of loading
- n_f = Total No. of frames in the direction of loading
- A_c = Summation of the cross- section area of all columns and shear wall in the storey under consideration
- V_j = Maximum Storey Shear at storey level 'j'

f) Shear Stress at Storey Levels

Storey	n_c	n_{f1}	n_{f2}	A_c	Storey Shears (KN)	Shear Stress	
						T_{col} 1(MPa)	T_{col} 2(MPa)
4	4	2	2	0.211	106.40	1.01	1.01
3	9	3	3	0.476	285.49	0.90	0.90
2	9	3	3	0.476	353.30	1.11	1.11
1	9	3	3	0.476	370.25	1.17	1.17

$T_{col} >>$ min of 0.4MPa and $0.1\sqrt{f_{ck}}$ = 0.45MPa

Hence, the check is not satisfied.

A.2.7. Case II: STRUCTURAL ANALYSIS OF THE BUILDING (After occupancy change to a Health Clinic)

A.2.7.1. Method I: As per NBC 105:1994

Major changes while converting Residential building into Health clinic

S.No	Description of Building	Live load(kN/m²)	Importance Factor
1.	Residential	2.5	1
2.	Health Clinic	3	1.5

Seismic evaluation of building under consideration.

Assumptions:

Unit weight of brick work = 19.6 KN/m^3

Live load = 3.0 KN/m^2

a) **LIVE LOAD CALCULATION**

LEVEL	FLOORS	FLOOR AREA sq.m	LL	0.25LL	Remarks
4	Third Floor	29.485	44.2275	11.056875	
3	Second Floor	90.33	270.99	67.7475	
2	First Floor	67.73	203.19	50.7975	
1	Ground Floor	67.73	203.19	50.7975	
∑				180.39938	

Note: Quick check calculation based on FEMA 310 for the seismic evaluation of building under consideration

b) **LUMP MASS CALCULATION**

S.NO.	FLOORS	Total Dead Load (KN)	Total Live Load (KN)	Total Weight (KN)	Remarks
4	Third Floor	260.48	11.056875	271.54	
3	Second Floorz	756.04	67.7475	823.79	
2	FirstFloor	649.82	50.7975	700.62	
1	Ground Floor	649.82	50.7975	700.62	
∑				2496.56	

Calculation Of Base Shear (Using NBC 105:1994)

c) Design Horizontal Seismic Coefficient for the Seismic Coefficient Method

The design horizontal seismic force coefficient, Cd shall be taken as :

$$Cd = C\ Z\ I\ K$$

Where, C is the basic seismic coefficient for the fundamental translational period in the direction under consideration.

Z = Seismic Zoning Factor   I = Importance Factor
K = Structural Performance Factor

The total design lateral force or Design Seismic Base Shear (VB) along any principal direction is determined by the following expression :

$$VB = CdWt$$

Where, Cd = The Design Horizontal Seismic Coefficient
Wt = Total of the gravity loads of the whole building

The approximate fundamental natural period of vibration (Ta) in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

NBC:105:1994 $Ta = (0.09h)/\sqrt{d}$

Where,
h = Height of Building in meter = 10.8m
d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force
dx = 8.23m
dz = 8.23m
Tax = $(0.09h)/\sqrt{dx}$ = 0.338
Taz = $(0.09h)/\sqrt{dz}$ = 0.338
Therefore ,C = 0.08 for medium soil

Seismic zoning factor for Lalitpur is, Z = 1.0
Cd = C Z I K
= 0.08 X 1 X 1.5 X 1 = 0.12

Base shear = V= CdWt
= 0.12×2468.34   296.20KN

d) Distribution Of Base Shear And Calculation Of Shear Stress In RC Columns

The horizontal seismic force at each level i shall be taken as :
The design base shear (V) computed shall be distributed along the height of the building as per the following expression:

$$Fi = V \times \frac{Wihi}{\sum Wihi}$$

Where,
Wi = Proportion of Wtcontributed by level i,
hi = Height of floor i measured from base

Floor	Total weight W_i (KN	Height h_i (m)	W_ih_i	$\frac{W_ih_i}{\sum W_ih_i}$	Qi (KN) $= V \times \frac{W_ih_i}{\sum W_ih_i}$	Storey Shear F_i (KN)
4.00	271.54	10.8	2932.63	0.19	56.27	56.27
3.00	823.79	8.1	6672.69	0.43	127.36	183.63
2.00	700.62	5.4	3783.34	0.24	71.08	254.71
1.00	700.62	2.7	1891.67	0.12	35.54	290.25
$\sum$	2468.34		15280.33	1.00	290.25	

(Using IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of Buildings, 6.5.1)

Average Shearing stress in columns is given as

$T_{col} = \frac{n_c}{n_c - n_f} \times \frac{V_j}{A_c} <$ min of 0.4MPa and $0.1\sqrt{f_{ck}}$ (Ref: IS 15988:2013 Clause 6.5.1)

$0.1\sqrt{fck} = 0.45$

For Ground Storey columns,

n_c = Total no. of Columns resisting lateral forces in the direction of loading
n_f = Total no. of frames in the direction of loading
A_c = Summation of the cross- section area of all columns in the storey under consideration
V_j = Maximum Storey shear at storey level 'j'
DCR = Demand Capacity Ratio

Floor	Total weight W_i (KN	Height h_i (m)	W_ih_i	$\frac{W_ih_i}{\sum W_ih_i}$	Qi (KN) $= V \times \frac{W_ih_i}{\sum W_ih_i}$	Storey Shear F_i (KN)
4.00	271.54	10.8	2932.63	0.19	56.27	56.27
3.00	823.79	8.1	6672.69	0.43	127.36	183.63
2.00	700.62	5.4	3783.34	0.24	71.08	254.71
1.00	700.62	2.7	1891.67	0.12	35.54	290.25
$\sum$	2468.34		15280.33	1.00	290.25	

T_{col} >min of 0.4 MPa and $0.1\sqrt{f_{ck}} = 0.45$MPa

Hence the check is not satisfied

A.2.7.2. Method II: As per IS 1893:2002-Part 1

Assumptions:

Unit weight of brick work = 19.6 KN/m^3
Live load = 3.0 KN/m^2

a) **LIVE LOAD CALCULATION**

LEVEL	FLOORS	FLOOR AREA (Sq.m)	LL	0.25LL	Remarks
4	Third Floor	29.485	44.2275	11.056875	
3	Second Floor	90.33	270.99	67.7475	
2	FirstFloor	67.73	203.19	50.7975	
1	Ground Floor	67.73	203.19	50.7975	
$\sum$				180.39938	

Note: Quick check calculation based on FEMA 310 for the seismic evaluation of building under consideration

b) **LUMP MASS CALCULATION**

S.NO.	FLOORS	Total Dead Load (KN)	Total Live Load (KN)	Total Weight (KN)	Remarks
4	Third Floor	260.48	11.056875	271.54	
3	Second Floor	756.04	67.7475	823.79	
2	FirstFloor	649.82	50.7975	700.62	
1	Ground Floor	649.82	50.7975	700.62	
$\sum$				2496.56	

c) **CALCULATION OF BASE SHEAR**

The total design lateral force or design seismic base shear is given by
Based on IS 1893 (Part 1): 2002, Criteria for earthquake resistant design of structures,
Calculation of earthquake loads using Seismic coefficient method:

The design horizontal seismic coefficient, $A_h = \frac{ZIS_a}{2R_g}$

Where Z = Zone Factor
I = Importance Factor
R = Response Reduction Factor
S_a/g = Average Response Acceleration Coefficient

The total design lateral force or Design Seismic Base Shear (V_B) along any principal direction is determined by the following expression :

$V_B = A_h W$

Where,

A_h = The Design Horizontal Seismic Coefficient
W = Seismic weight of the building

The approximate fundamental natural period of vibration (Ta) in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

$$Ta = \frac{0.09h}{d^{0.5}}$$ Ref: IS:1893:2002(part1)

Where,

h = Height of Building in meter = 10.80 m

d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force

dx = 8.23 m

dz = 8.23 m

$T_{ax} = \dfrac{0.09h}{dx^{0.5}}$

=0.338 <0.55

$T_{az} = \dfrac{0.09h}{dx^{0.5}}$

=0.338 <0.55

Therefore,

Sa/g = 2.5 for medium soil (IS :1893(Part 1) : 2002

Z = 0.36 (For Seismic Zone V) (Refer IS 1893 (Part 1) :2002-table 2)

I = 1.50 (For Clinic Building) (Refer IS 1893 (Part 1) :2002-table 6)

Sa/g = 2.5 (For Medium Soil) (Refer IS 1893 (Part 1) :2002-Clause 6.4.5 and Fig.2)

R = 3.0 (For Ordinary RC Moment Resisting Frame)(Ref. IS 1893 (Part 1) :2002-table 7)

$Ah = \dfrac{ZIS_a}{2Rg}$

= 0.36 ×1.5 × 2.5/ 2 × 3

= 0.225

Base shear = $V_b = A_h W$

= 0.225 × 2496.56

= 561.726 KN

d) Distribution of Base Shear and Calculation of Shear Stress in RC Columns :

Floor	Total weight W_i (KN)	Height h_i (m)	$W_i h_i^2$	$\dfrac{W_i h_i^2}{\sum W_i h_i^2}$	Q_i(KN)	Storey Shear V_i (KN)
4.00	271.54	10.8	31672.06	0.28	159.91	159.91
3.00	823.79	8.1	54048.70	0.49	272.88	432.79
2.00	700.62	5.4	20430.01	0.18	103.15	535.94
1.00	700.62	2.7	5107.50	0.05	25.79	561.73
$\sum$	2496.56		111258.27	1.00	561.73	

e) SHEAR STRESS AT STOREY LEVEL :

(Using IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of buildings 6.5.1)

The Total design lateral force or design seismic base shear is given by

$V_b = A_h W$

Average Shearing stress in columns is given as

(Ref: IS 15988:2013 Clause 6.5.1)

$T_{col} = \left(\frac{n_c}{n_c - n_f}\right) \times \left(\frac{v_j}{A_c}\right)$ < min. of 0.4 Mpa and $0.1\sqrt{f_{ck}}$

For Ground Storey columns,

n_c = Total No. of Columns resisting lateral forces in the direction of loading

n_f = Total No. of frames in the direction of loading

A_c = Summation of the cross- section area of all columns and shear wall in the storey under consideration

V_j = Maximum Storey Shear at storey level 'j'

f) Shear Stress at Storey Levels

Storey	n_c	n_{f1}	n_{f2}	A_c	Storey Shears (KN)	Shear Stress	
						T col 1(MPa)	T col 2(MPa)
4	4	2	2	0.211	159.91	1.52	1.52
3	9	3	3	0.476	432.79	1.36	1.36
2	9	3	3	0.476	535.94	1.69	1.69
1	9	3	3	0.476	561.73	1.77	1.77

T_{col}>>min of 0.4 MPa and $0.1\sqrt{f_{ck}} = 0.45\text{MPa}$

Hence, the check is not satisfied.

Since columns are not safe, now checking for different categories as below:

g) Calculation of Shear Capacity of Colum Using Capacity design Method :

- Checking Shear Capacity of Center Column :
-

Shear Capacity of column required $= 1.4\,\frac{M_1 + m_1'}{h_{st}}$ (Ref:IS 13920:1993 Clause 7.3.4)

The longitudinal Beam size is equal to 230 × 350.

Reinforcement of Beam is equal to 3 TOR 16 top and bottom.

Where,

b = 230

D = 350

d $= 350 - 25 - \frac{16}{2}$ = 317

The Moment Capacities are evaluated from STAADPro 2006, which is equal to 68.6 KN-m and 53.6 KN-m.
Shear force in Column corresponding to these moments :
(Ref:IS 13920:1993 Clause 7.3.4)

$$V_u = 1.4\frac{M_1 + m_1'}{h_{st}}$$

$$= 1.4 \times \frac{68.6 - 53.6}{2.7}$$

$$= 63.36 \quad KN$$

Size of Column = 230 mm × 230 mm
Area of Steel (A_{st}) = 4 tor- 12 diameter
F_{ck} = 20 N/mm²
F_y = 415 N/mm²

From SP 16 Table 61
forP_t = 0.85% ,τ = 0.585 N/mm²
Shear Capacity = 0.585 ×230 × 230/1000
= 30.94 kN

Shear to be carried Stirrups V_{us} = 63.36 – 30.94
= 32.42 KN

From SP 16 Table 62 :
Stirrups in the Column : Tor 8 Diameter @150 mm c/c

$\frac{V_{us}}{d}$ = 2.42 KN/ cm

V_{us} = 2.42 × 19.2 kN/ cm
= 46.5 KN>>32.42 KN

Hence, the Check for shear tie is satisfied for central column.

h) Axial Stress Check:

1. ***The Axial Stress due to Gravity Loads as per FEMA 310***

Permissible axial stress = $0.1f_c'$ = 2 N/mm²
The axial stress due to gravity loads in the center column of Ground Floor =
The axial stress due to gravity loads in column

= Load on column(N) / Cross section Area of Column = $\frac{711.289 \times 1000}{230 \times 230}$
= 13.446 N/mm² > 2 N/mm²

Hence the check not satisfied.

2. ***Axial stresses due to overturning forces as per FEMA 310***

2.1 Axial stress in moment frames for x-direction loading

Axial force in columns of moment frames at base due to overturning forces,
The axial stress of columns subjected to overturning forces F_o is given by

From Fema 310 clause 3.5.3.6

$$\mathbf{F_o} = \frac{2}{3}\left(\frac{V_b}{n_f}\right) \times \left(\frac{H}{L}\right)$$

V_b = Base shear x Load Factor

561.726 x1.5 = 842.59 KN

A_c = column area = 0.0529 sq.m.

H=total height = 10.8 M

L=Length of the building = 8.00 M

From Fema 310 clause 3.5.3.6

$$\mathbf{F_o} = \frac{2}{3}\left(\frac{V_b}{n_f}\right) \times \left(\frac{H}{L}\right)$$

= 252.78 KN

Axial Stress for x-direction loading,

σ = 252.78/0.05 = 4.78 MPa

σ_{all} = 0.25 fck (Ref:IS 15988:2013 clause 6.5.4) = 5.00 MPa

Therefore $\sigma < \sigma_{all}$ OK

DCR = 0.334

Hence the check is satisfied.

2.2 Axial stress in moment frames for z-direction loading

Axial force in columns of moment frames at base due to overturning forces,

The axial stress of columns subjected to overturning forces Fo is given by

From Fema 310 clause 3.5.3.6

$$\mathbf{F_o} = \frac{2}{3}\left(\frac{V_b}{n_f}\right) \times \left(\frac{H}{L}\right)$$

V_b	= Base shear x Load Factor	
	= 561.726 x1.5	= 842.59 KN
A_c	= column area	= 0.0529 sq.m.
H	=total height	= 10.8 M
L	=Length of the building	= 8.00 M

From Fema 310 clause 3.5.3.6

$$\mathbf{F_o} = \frac{2}{3}\left(\frac{V_b}{n_f}\right) \times \left(\frac{H}{L}\right)$$

= 252.78 KN

Axial Stress for x-direction loading,

σ = 252.78/0.05 = 4.78 MPa

σ_{all} =0.25 f_{ck}(Ref:IS 15988:2013 clause 6.5.4) = 5.00 MPa

therefore $\sigma < \sigma_{all}$ OK

DCR = 0.334

Hence the check is satisfied

i) Check for Out-of-Plane Stability of Brick Masonry Walls

Wall Type	Wall Thickness	Recommended Height/ Thickness ratio (0.24 <Sx ≤ 0.35) (Ref: Fema 310 Table 4-2)	Actual Height/ Thickness ratio in building	Comments
Wall in ground storey	230 mm	1\8	$\frac{2700 - 350}{230} = 10.217$	Pass
Wall in upper stories	230 mm	16	$\frac{2700 - 350}{230} = 10.217$	Pass

Hence the check is satisfied.

A.2.8. RETROFITTING DRAWINGS

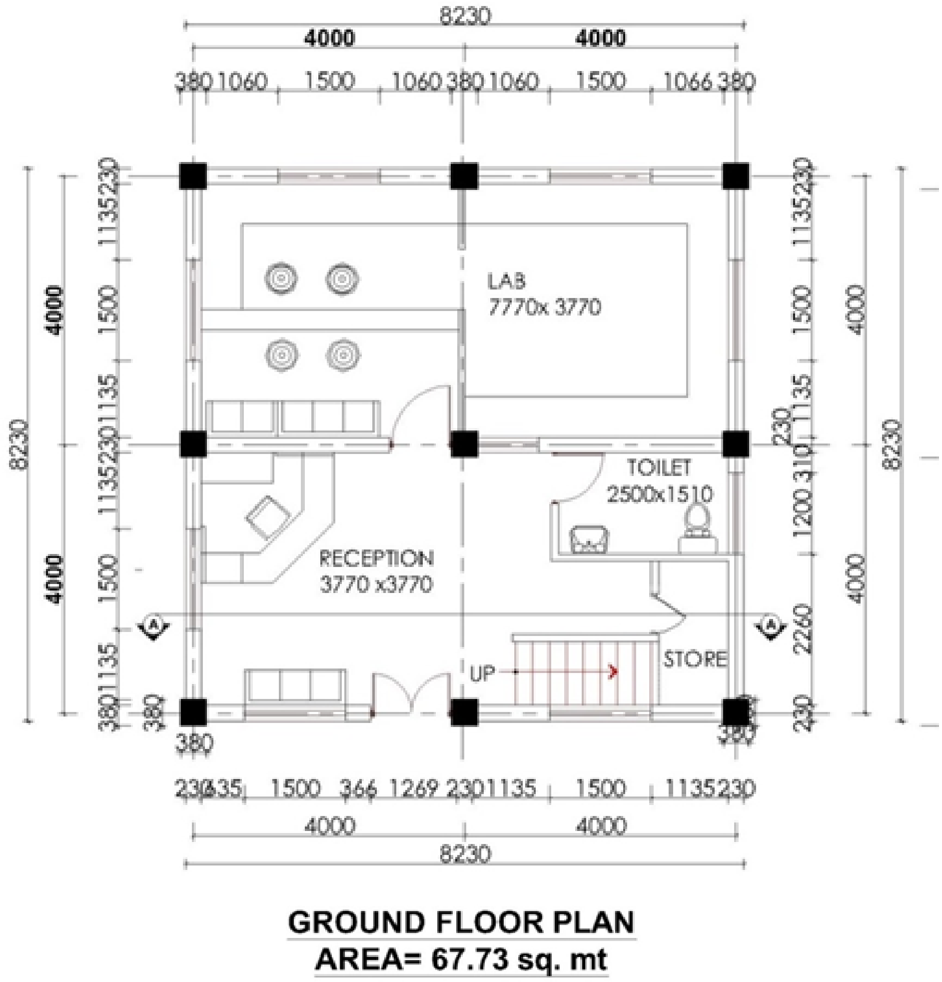
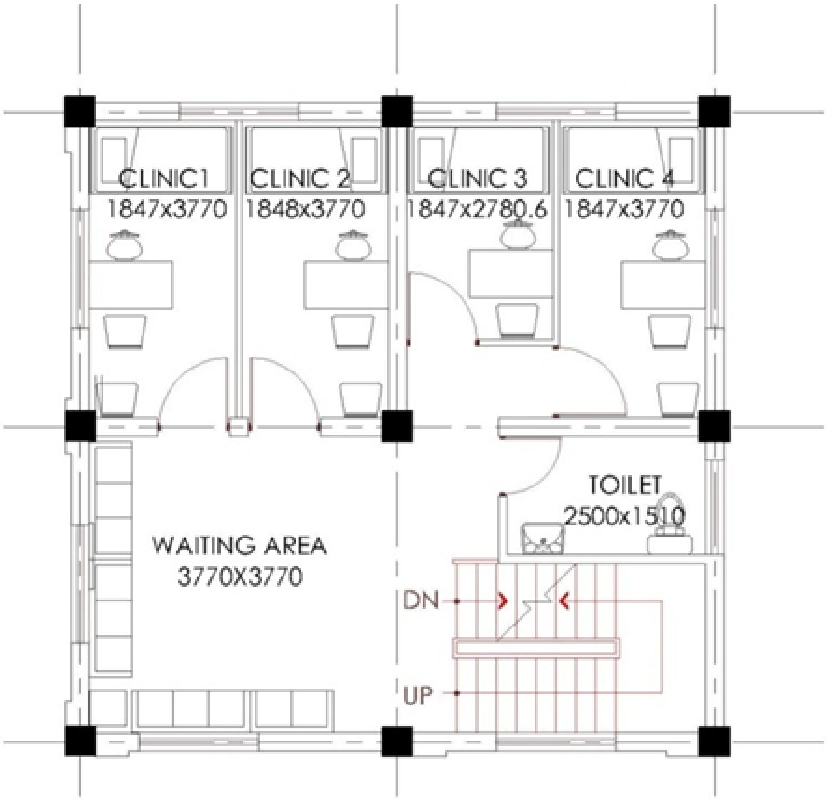
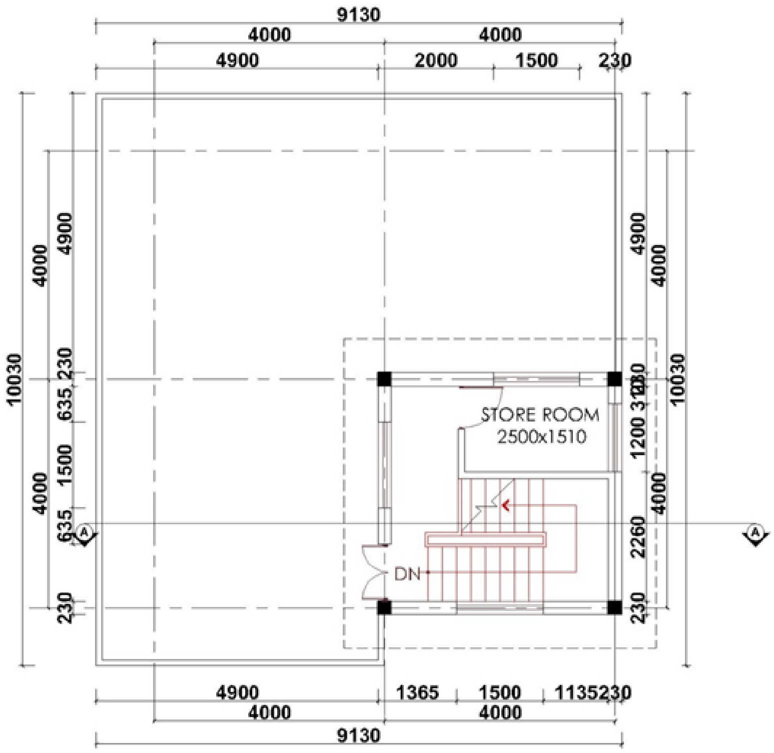


Figure A-11 Retrofitted ground floor plan



FIRST FLOOR PLAN
AREA= 67.73 sq. mt



TOP FLOOR PLAN

Figure A-12 Retrofitted first and top floor plan

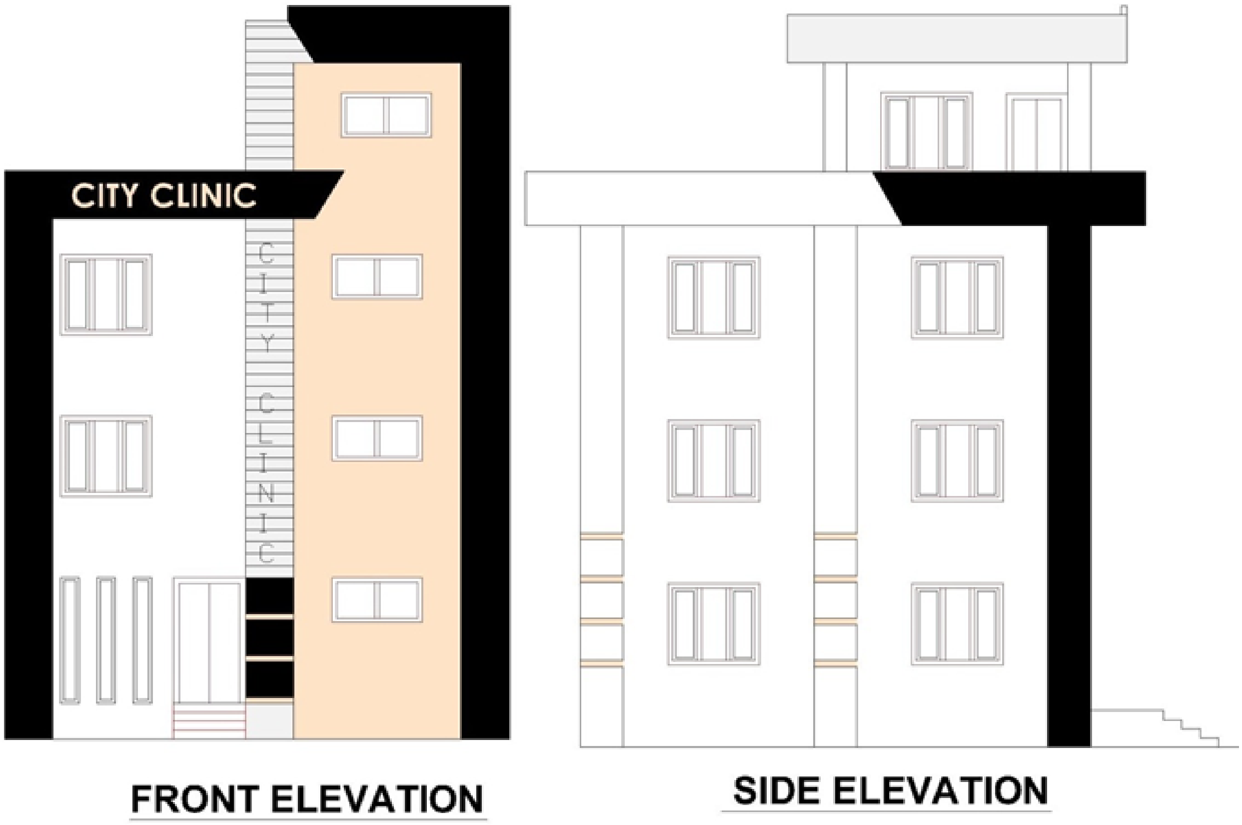


Figure A-13 Front and side elevation

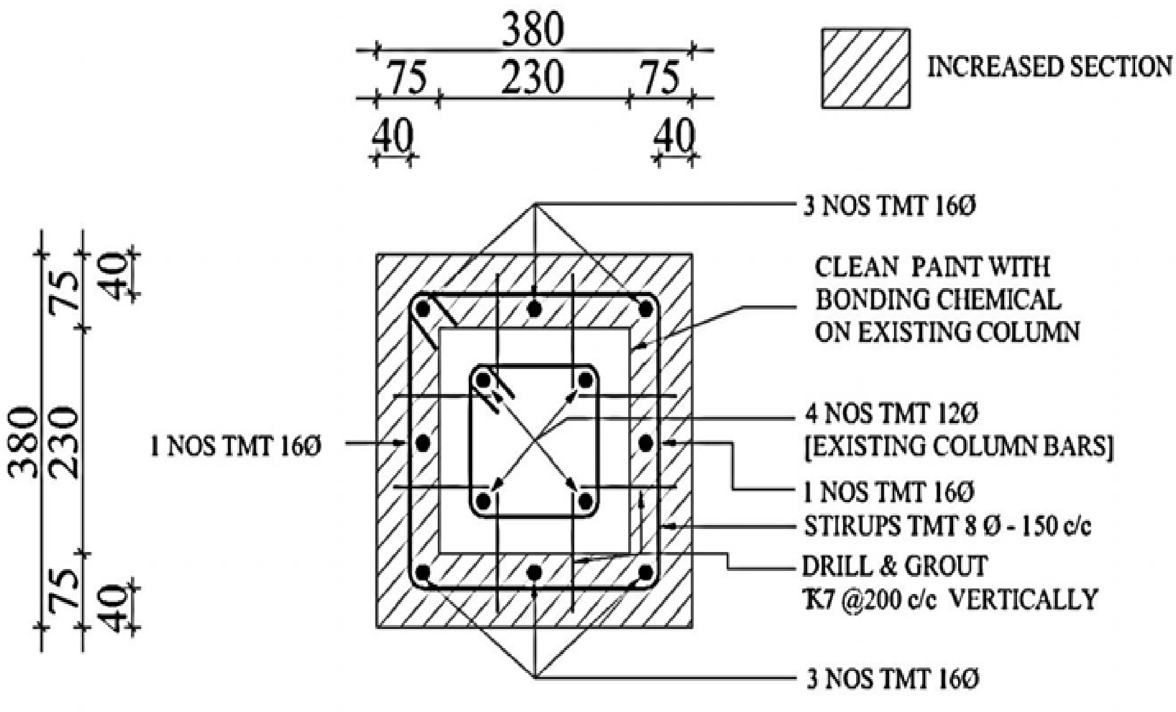


Figure A-14 Section of jacketed column C1

EXAMPLE 3: STRESS CHECK CALCULATION USING FEMA 310

The following is a sample of quick check calculations based on FEMA 310 for the seismic evaluation of building under consideration.

Assumptions:

Unit weight of RCC = 24 KN/m^3; Unit weight of brick = 19 KN/m^3

Live load = 3 KN/m^2; Live load at roof level without access= 1.5 KN/m^2

Weight of plaster and floor finish = 1.0 KN / m^2

Weight of timber = 6.45 KN/m^2

Lateral load is solely carried by frame elements. Stiffness of the walls is not considered.

- Grade of concrete = M20 for all structural elements
- Grade of steel = Fe 415

A.2.9. Calculation for Shear Stress check

Summary of lumped load calculation

LOAD	MULTIPLIER
Dead	1
Live	0.25

Total mass for seismic weight calculation W = 9661.32 KN

Calculation of base shear (Using IS 1893 (Part I):: 2002)

Based on IS 1893 (Part 1): 2002, Criteria for earthquake resistant design of structures, Calculation of earthquake loads using Seismic coefficient method:

The design horizontal seismic coefficient, $A_h = \frac{ZIS_a}{2Rg}$

Where Z = Zone factor

I = Importance factor

R = Response reduction factor

$\frac{S_a}{g}$ = average response acceleration coefficient

The approximate fundamental natural period of vibration (T_a) in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

$$T_a = \frac{0.09h}{d^{0.5}}$$

Where,

h = Height of Building in meter and

d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force

I = 1 (from clause 6.4.2, IS 1893 (Part 1) 2002)

Z = 0.36

$A_h = \frac{ZIS_a}{2Rg}$

$T_a = \frac{0.09h}{dx^{0.5}}$

$= \frac{0.09 \times 8.64}{29.53^{0.5}}$ = 0.1431 sec

$T_{ay} = \frac{0.09h}{dy^{0.5}}$

$\frac{0.09 \times 8.64}{7.92^{0.5}}$ = 0.2763 sec

$\frac{S_a}{g}$ = 2.5 (from graph 1893 (part 1)-2002)

R = 5

$A_h = \frac{ZIS_a}{2Rg}$

$= \frac{0.36 \times 1 \times 2.5}{2 \times 5}$ = 0.09

The total design lateral force or design seismic base shear (V_B) along any principal direction is determined by the following expression

$V_B = A_h W$

Where,

A_h = The design horizontal seismic coefficient

W = Seismic weight of the building

 V_B = 0.09 x 9661.32 = 869.52 KN

Distribution of base shear and calculation of storey shear

The design base shear (V_b) is distributed along the height of the building as per the following expression:

$$Q_i = V_b \times \frac{W_i h_i}{\sum W_i h_i}$$

Where

Q_i = Design lateral force at floor i

W_i = Seismic weight of floor i

h_i = Height of floor i measured from base

STORY	STOREY FORCE Q_i (KN)	STOREY SHEAR V_j (KN)
3FL	353.9	353.9
2FL	337.94	691.84
1FL	177.68	869.52

Level	Storey Shears		A_c	n_c	n_{f1}	n_{f2}	n_c-n_{f1}	n_c-n_{f2}	Shear Stress	
	V_j(KN)	V_j (lb)	(in^2)						V_{1avg}(psi)	V_{2avg}(psi)
3	353.9	79287.6	3991	41	11	4	30	37	13.6	11.01
2	691.84	154999.5	4158	41	11	4	30	37	25.47	20.65
1	869.52	194806.9	4747	45	11	5	34	40	27.16	23.08

Where,

A_c = Summation of the cross sectional area of all columns in the storey under consideration

n_c = Total no. of columns

n_f = Total no. of frames in the direction of loading

V_{avg} = Average shear stress (psi) in the columns of concrete frames

$$V_{avg} = \frac{1}{m}\left(\frac{n_c}{n_c - n_f}\right)\left(\frac{V_j}{A_c}\right)$$ (Ref: Fema 310 Clause 3.5.3.2)

m = component modification factor = 2 for buildings being evaluated to the life safety performance level

f'_c = specified compressive strength of concrete = 20 N/mm^2

The average induced shear stresses are less that the permissible value of 100 psi or 2√fc' (107.59 psi).
Hence, safe.

Calculation of Shear capacity of column using capacity design method

Checking shear capacity of column (E-3)

Shear capacity of column (E-3) required $= \frac{1.2(M^l+M^r)}{h_{st}}$ (Ref: IS13920:2013 Clause 7.3.4)

Ultimate capacity of beam = $f_y \times A_{st} \times d_{eff}$(Ref: IS 456:2000 ANNEX G)

Where,

$d_{eff} = d\left\{\frac{1-(A_{st}-A_{sc})f_y}{f_{ck}bd}\right\}$ (Ref: IS 456:2000 ANNEX G)

Calculation of M^l

A_{st} = 226 mm^2 (3-16;3-12) ; b = 230mm ; d = 200mm

A_{sc} = 226 mm^2 ; f_{ck} = 20 N/mm^2

f_y = 415 N/mm^2 ; P_t = 0.49 %

$d_{eff} = 200\left\{\frac{1-(226-226)\times 415}{20\times 230\times 200}\right\}$

d_{eff} = 200 mm

Beam moment capacity M^l = $415 \times 226 \times 200$ Nmm
= 18.76 KN –m = M^2

Hence, Shear capacity of column (E-3) required = $\frac{1.2(18.76+18.76)}{2.88}$

= 15.63 KN

P_t provided $= \frac{1206\times100}{230\times350}$ = 1.5 %

From table 61, for P_t = 1.5 %, M20 concrete τ_c = 0.72 N/mm²

Shear capacity of concrete section = $\frac{0.72\times230\times310}{1000}$ = 51.34 KN>15.63KN

Hence safe.

Checking shear capacity of column (D-3)

Shear capacity of column (D-3) required $= \frac{1.2(M^l+M^r)}{h_{st}}$

Ultimate capacity of beam = $f_y \times A_{st} \times d_{eff}$

Where,

d_{eff} = $d\left\{\frac{1-(A_{st}-A_{sc})f_y}{f_{ck}bd}\right\}$ (Ref: IS 456:2000 ANNEX G)

Calculation of M^l

A_{st} = 427 mm² (3-16;3-12) ; b = 230mm ; d = 350mm
A_{sc} = 226 mm² ; f_{ck} = 20 N/mm²
f_y = 415 N/mm² ; P_t = 0.53 %

d_{eff} = $350\left\{\frac{1-(427-226)\times415}{20\times230\times350}\right\}$
d_{eff} = 332 mm

Beam moment capacity M^l = $415 \times 427 \times 332$ Nmm
= 58.83 KN –m

$M^2 = 0$
Hence, Shear capacity of column (E-3) required $= \frac{1.2(58.83+0)}{2.88}$

= 24.51 kN

P_t provided = $\frac{1206\times100}{230\times350}$ = 1.5 %

From table 61, for P_t = 1.5 %, M20 concrete τ_c = 0.72 N/mm²

Shear capacity of concrete section $= \frac{0.72\times230\times310}{1000}$ = 51.34 kN>24.51kN
Hence safe.

Axial Stress check

Axial stresses due to overturning forces as per FEMA 310

Permissible stress = 868 psi (0.3 f_c')

The axial stress of columns subjected to overturning forces pot is given by

$$P_{ot} = \left(\frac{1}{m}\right)\left(\frac{2}{3}\right)\left(\frac{Vh_0}{Ln_f}\right)\left(\frac{1}{A_c}\right)$$

Where,

n_f = Total no. of frames in the direction of loading = 5

V = Base shear = 869.52 KN = 194806.9 P

h_n = height (in feet) above the base to the roof level = 28.8ft

L = Total length of the frame (in feet) =98.43ft.

m = component modification factor = 2

A_c = Summation of the cross sectional area of all columns in the storey under consideration = 4747 in^2

P_{ot} = 0.8psi << 868 psi

Hence Safe

Check for torsion

Checking eccentricity between centre of mass and centre of stiffness at different floors

STORY	CENTER OF MASS		CENTER OF STIFFNESS		% ECCENTRICITY		WIDTH	
	X_m	Y_m	X_r	Y_r	Ex	Ey	L	B
ROOF	14.764	5.295	14.764	5.7	0.00	-4.19	29.528	9.673
2FL	14.764	5.297	14.764	5.643	0.00	-3.58	29.528	9.673
1FL	14.764	5.215	14.764	5.565	0.00	-3.62	29.528	9.673

Check for Strong column weak beam

Checking capacity of column E-3 at ground floor

Ultimate capacity of beam = $f_y \times A_{st} \times d_{eff}$

Beam moment capacity M^1 = 415 ×226 × 200 N-mm

= 18.76 KN –m = M^2

Column moment capacity required = 1.2 × 18.76 = 22.51 KN –m

Column axial load = 364 KN (factored)

$$\frac{P_u}{f_{ck}bD} = \frac{364\times1000}{20\times230\times350} = 0.23$$

$$\frac{M_u}{f_{ck}bD^2} = \frac{22.51\times10^6}{20\times230\times350^2} = 0.04$$

Using SP – 16, P_t required is 0.15 % whereas P_t provided is 1.5 % .

Hence, the strong column-weak beam criteria meet.

Checking capacity of column D-3 at ground floor

Ultimate capacity of beam = $f_y \times A_{st} \times d_{eff}$

Beam moment capacity M^1 = $415 \times 427 \times 332$ Nmm

= 58.83 KN –m = M^2

Column moment capacity required = $1.2 \times 58.83 = 70.6$ KN –m

Column axial load = 418 KN (factored)

$$= \frac{P_u}{f_{ck} b D} = \frac{418 \times 1000}{20 \times 230 \times 350} = 0.26 \qquad \frac{M_u}{f_{ck} b D^2} = \frac{70.6 \times 10^6}{20 \times 230 \times 350^2} = 0.125$$

Using SP – 16 ,P_t required is 1.6 % whereas P_t provided is 1.5. %

Hence, the strong-column weak-beam criterion does not meet.

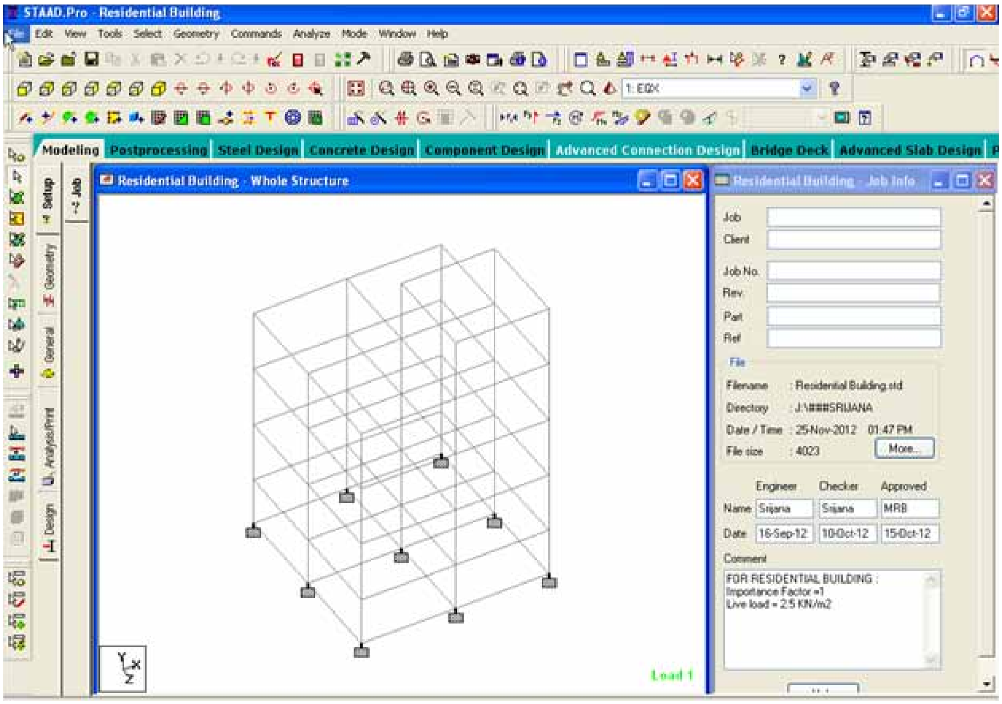
Check for out-of-plane stability of brick masonry walls

Wall type	Wall thick ness	Recommended Height/ Thickness ratio (0.24<Sx≤0.35) (Ref: Fema 310 Table 4-2)	Actual Height/ Thickness ratio in building	Comments
Wall in first storey,	230 mm	18	2530/230=11	Pass
	115 mm	18	2530/115 = 22	Fail
All other walls	230 mm	16	2530/230=11	Pass
	115mm	16	2530/115 = 22	Fail

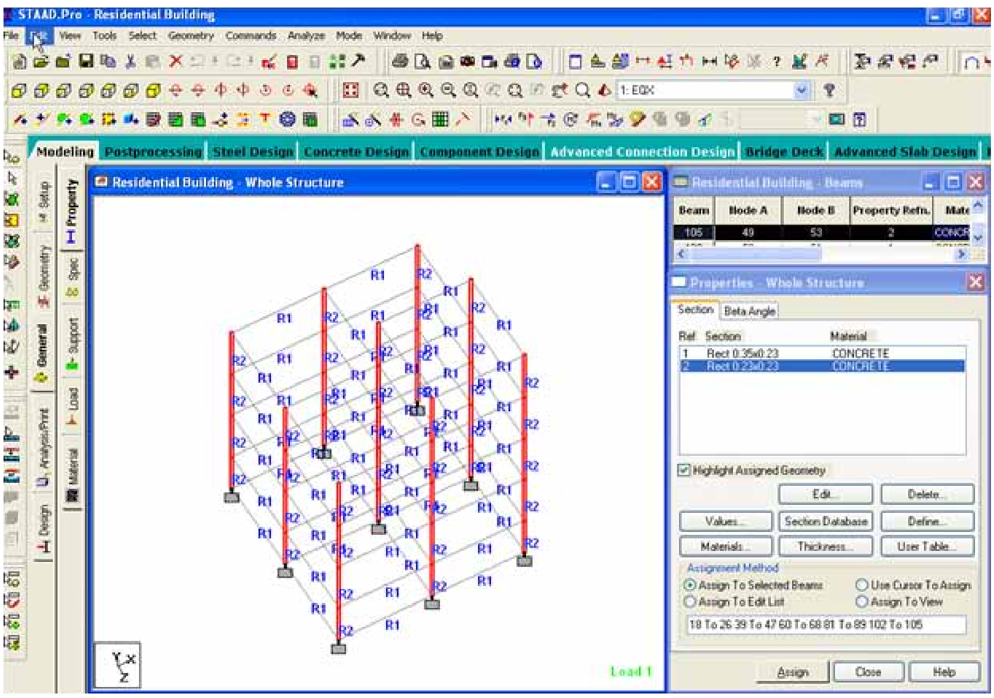
EXAMPLE 4: ANALYSIS OF BUILDING OF EXAMPLE 2 USING STRUCTURAL ANALYSIS PROGRAM-STAAD (STRENGTH BASED APPROACH)

Strength based Analysis of the structure with the given drawings was carried out for structural evaluation. The strength of the existing structure is evaluated and compared with the demand of the structure then structural members are retrofitted to fulfill the demand on the basis of strength.

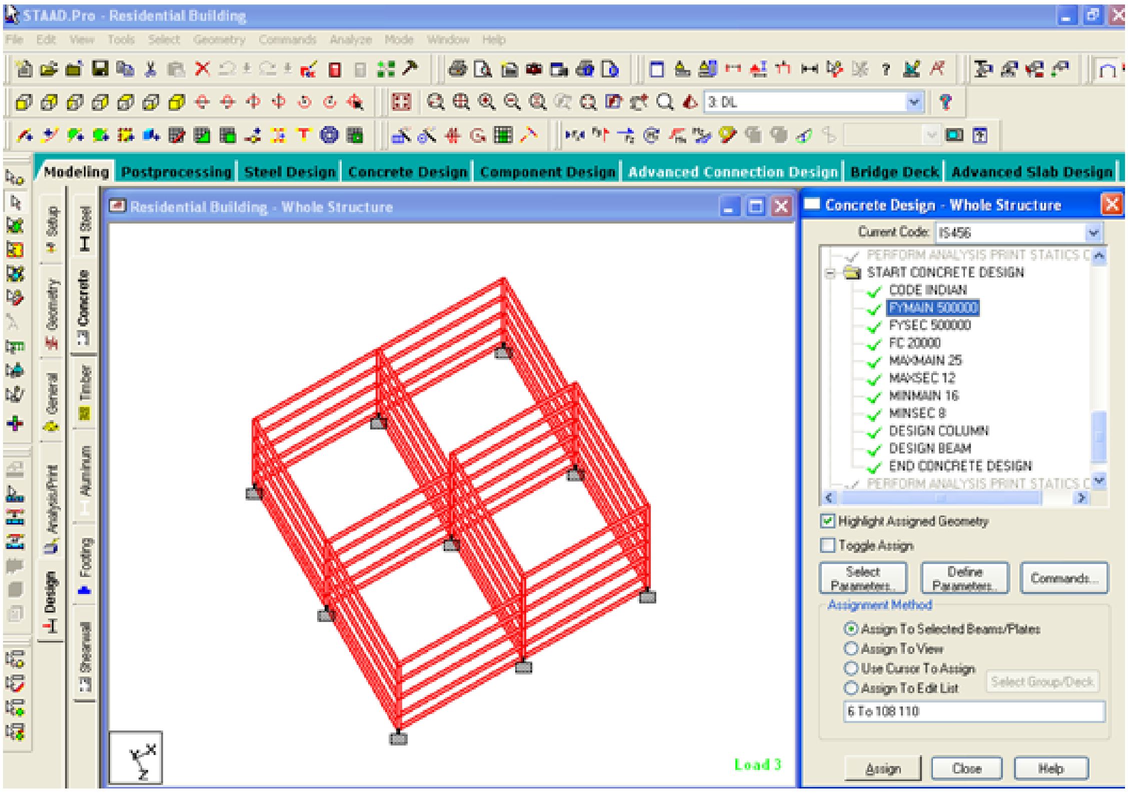
ANALYSIS IN STAAD



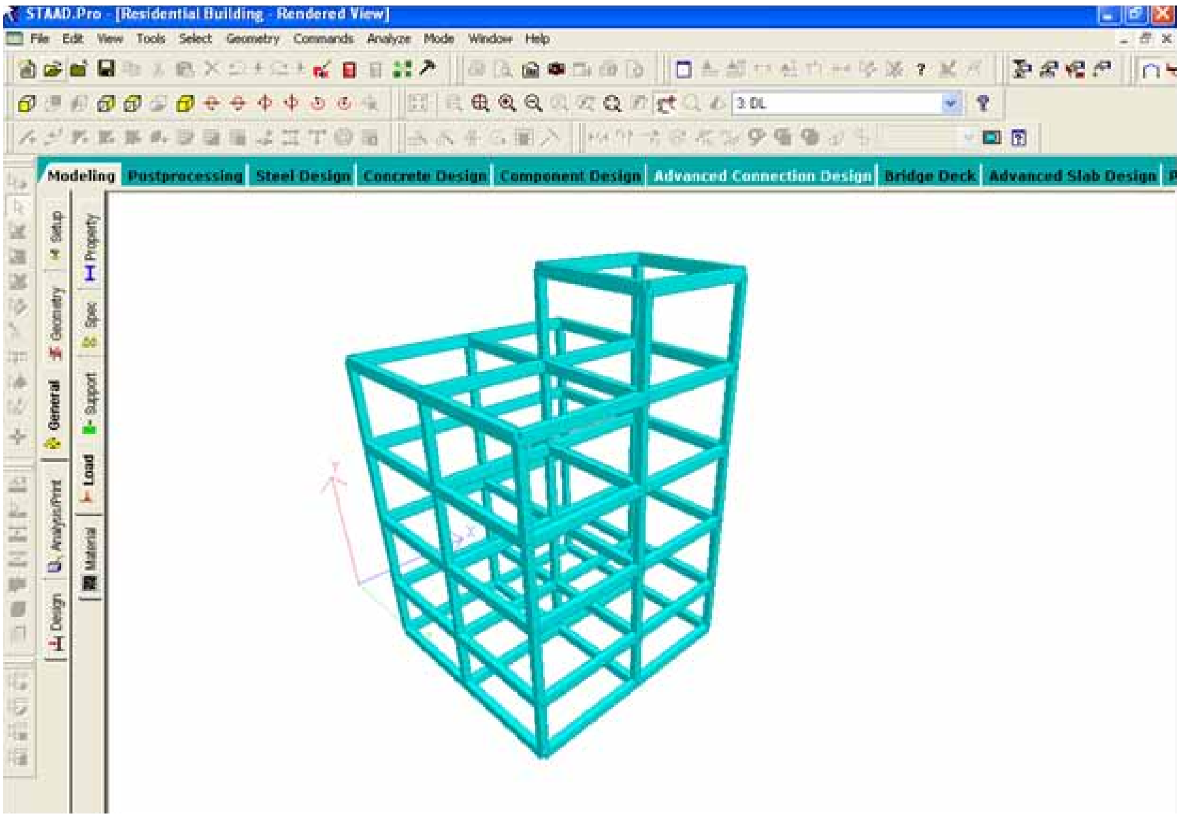
WIRE FRAME



COLUMN

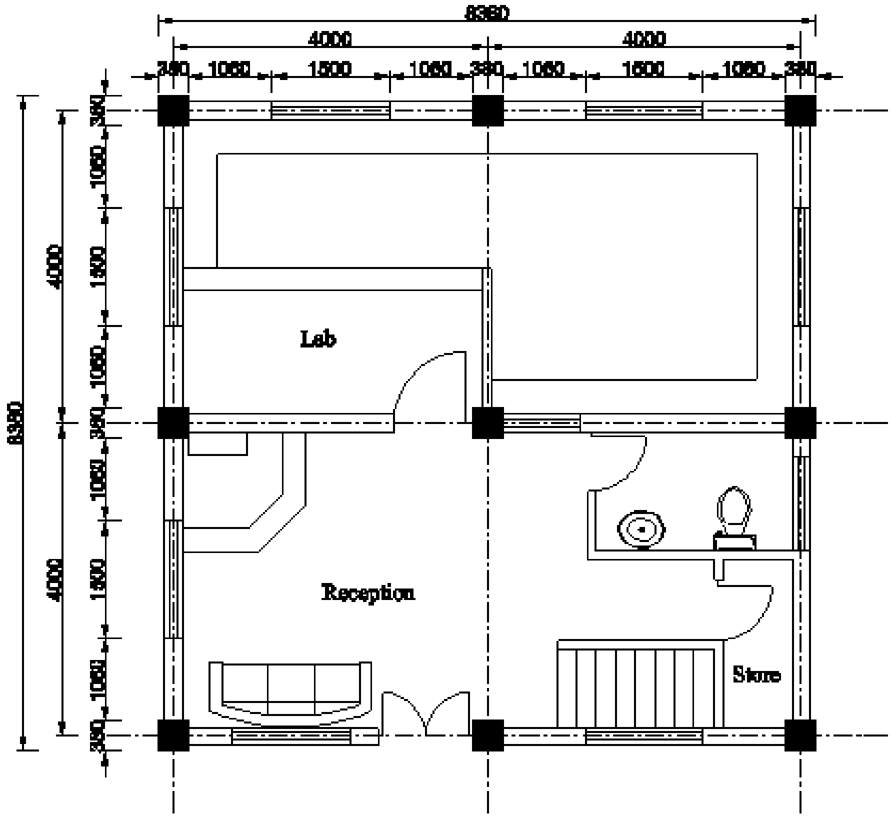


BEAM

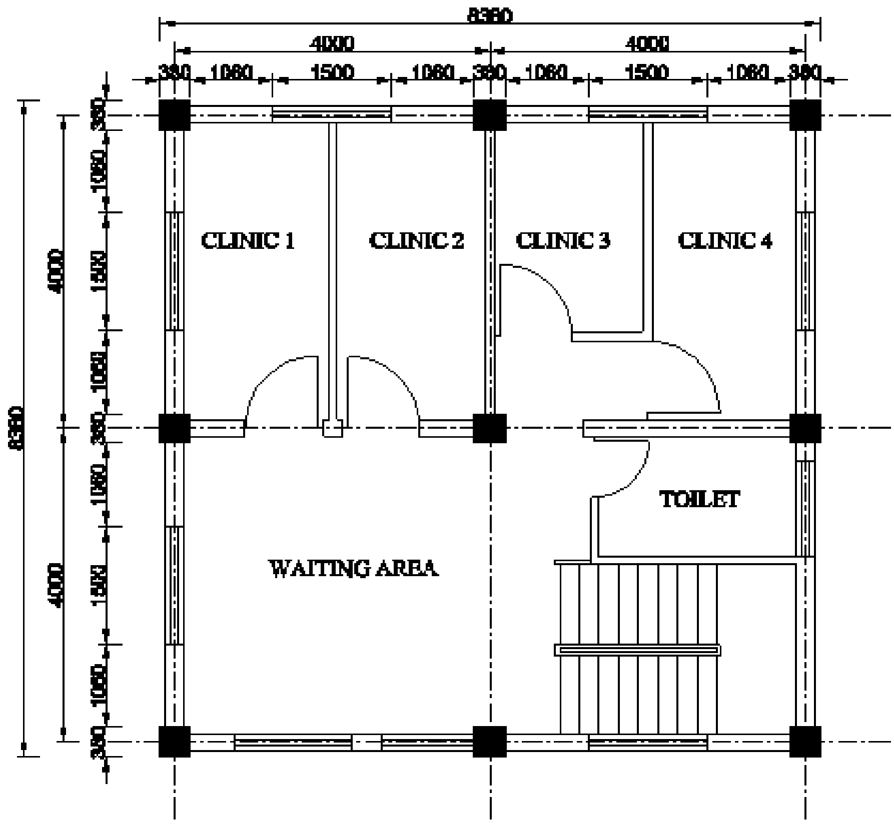


3D Model in Staad

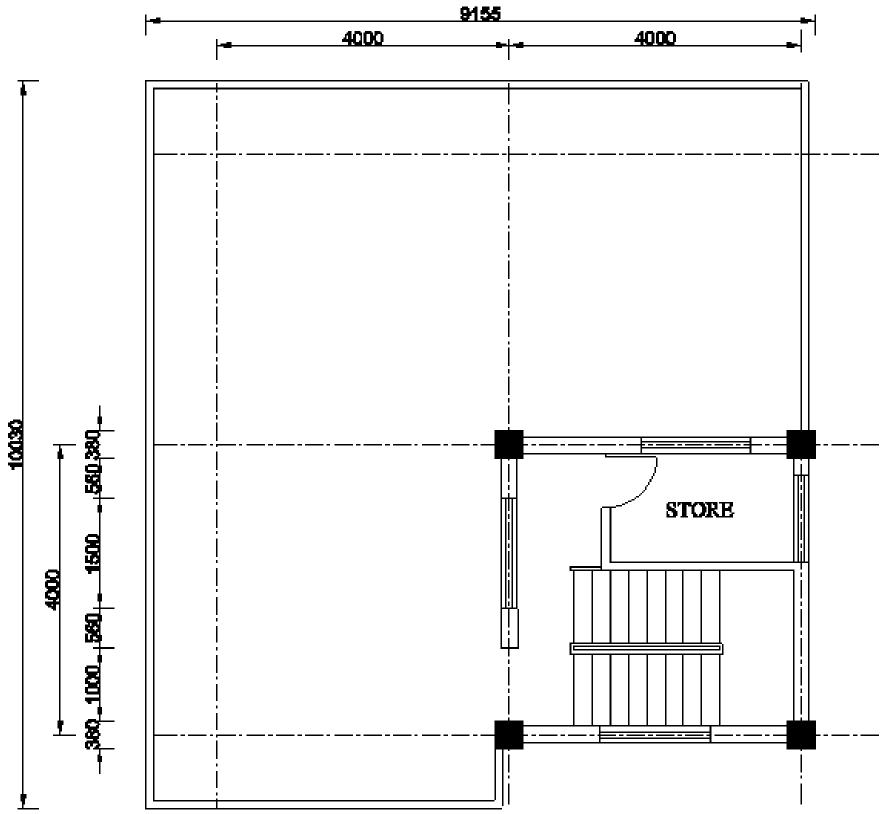
RETROFITTING DRAWINGS:



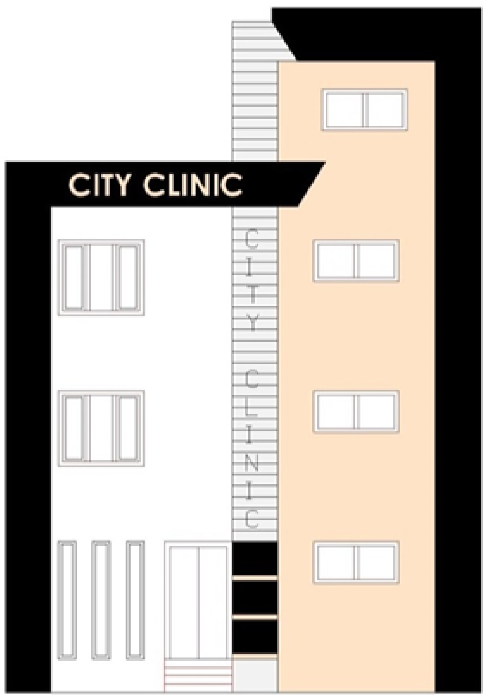
Ground Floor Plan



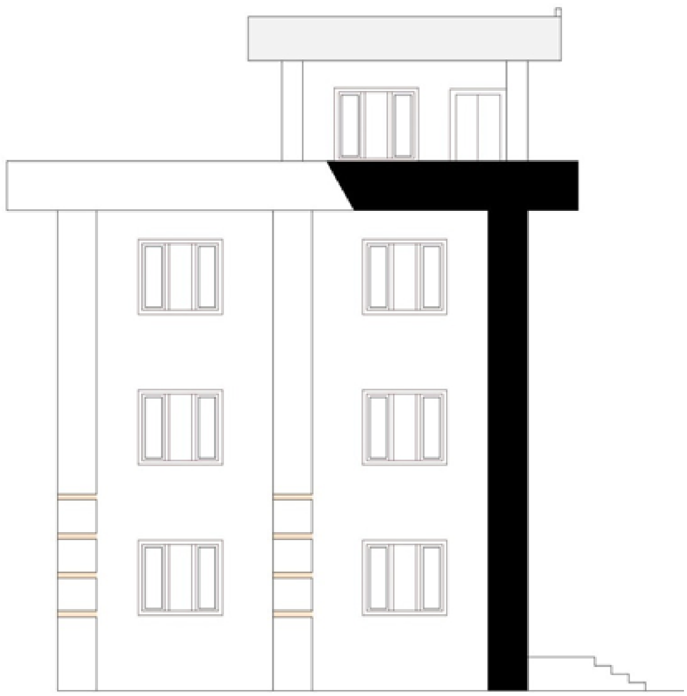
First Floor Plan



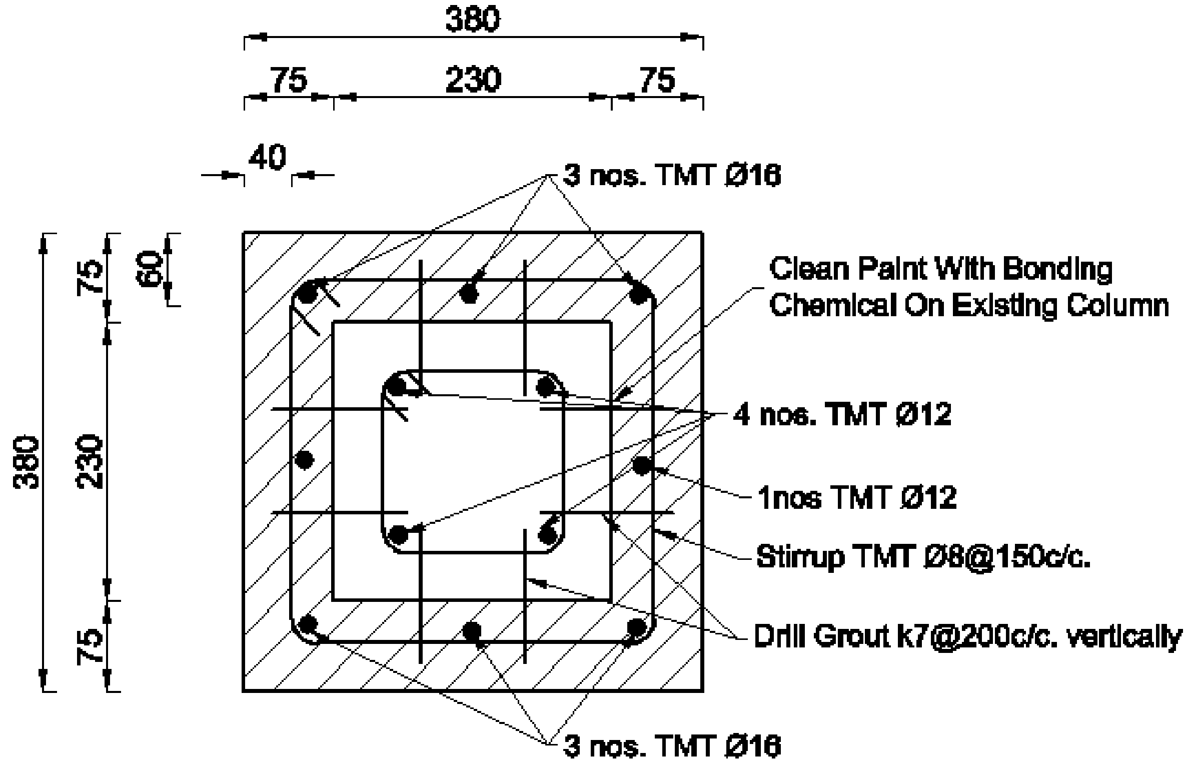
Top Floor Plan



FRONT ELEVATION



SIDE ELEVATION



SECTION OF JACKETTED COLUMN C1

EXAMPLE 5-A: PERFORMANCE ANALYSIS USING NON-LINEAR STATIC ANALYSIS (STATIC PUSHOVER ANALYSIS) OF BUILDING IN EXAMPLE -2, BEFORE RETROFITTING

Performance Analysis of the structure with the given drawings was carried out for structural performance evaluation. The performance evaluation of non-structural components and the combined performance is not evaluated. Static Pushover Analysis is used to evaluate the performance of the structure.

A.2.10. OUTPUT

The analysis is carried out for Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) as defined in IS 1893 (2002). The main results for both DBE and MCE are given below.

A.2.11. PERFORMANCE OF EXISTING STRUCTURE

The existing structure can't survive the Design Basis Earthquake as well as Maximum Considered Earthquake (MCE) since its capacity is low than the Demand.

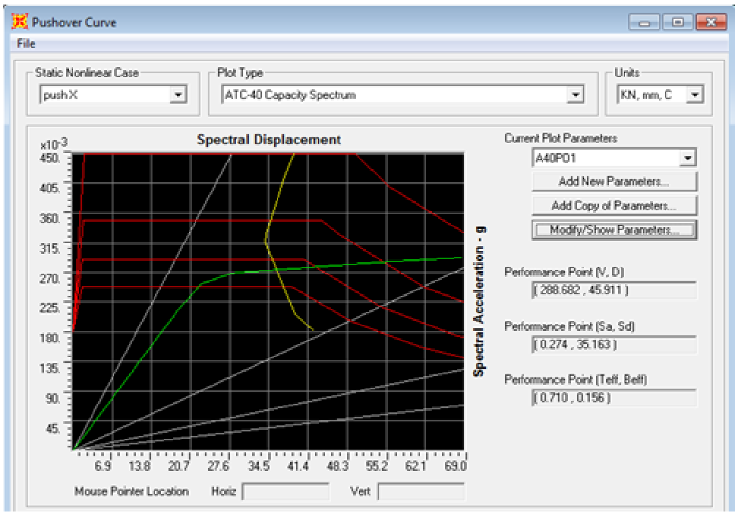


Figure A-15 Capacity Spectrum of Existing Structure at DBE

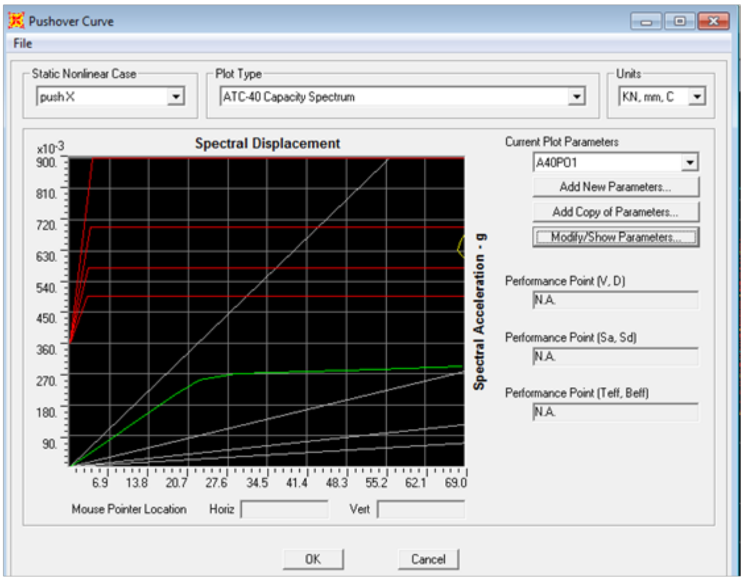
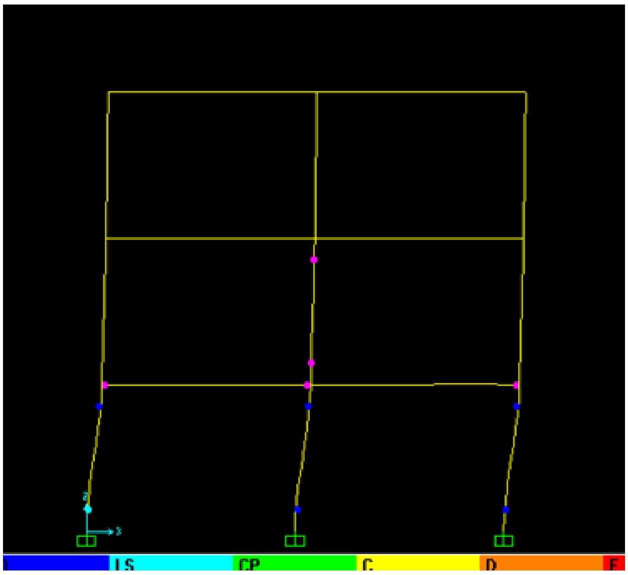
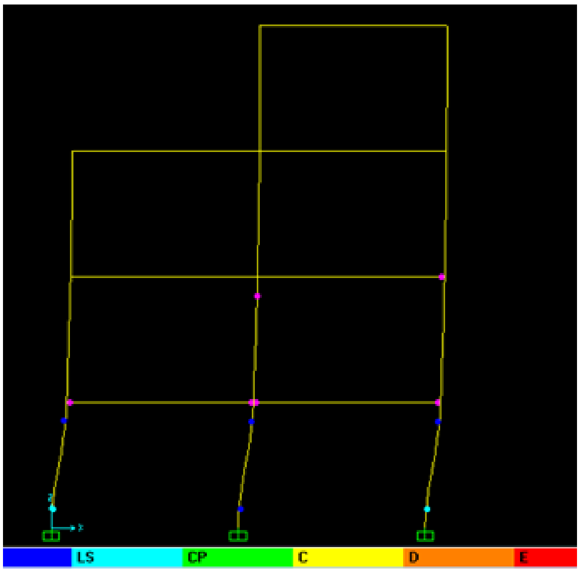


Figure A-16 Capacity Spectrum of Existing Structure

A.2.12. PLASTIC HINGES MECHANISM

Plastic hinge formation for the building mechanisms have been obtained at different displacements levels. The hinging patterns are as shown in figures

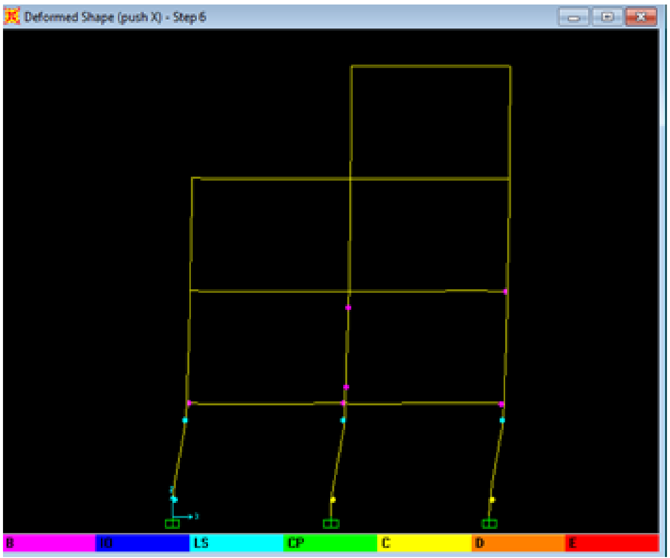




Building I

Plastic hinge (push X) of Grid 1-1

Plastic hinge (push X) of Grid 2-2



Plastic hinge (push X) of Grid 3-3

A.2.13. FINDINGS AND RECOMMENDATION

The analysis of the building was carried out using the static analysis and push over analysis in SAP 2000 and the buildings are found to be unsafe.

- Demand base shear is 516.726KN but base shear obtain is less than required in both Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE)

Hence the building cannot be considered safe for the hospital purpose. It needs retrofitting measures to increase capacity for the earthquake safety.

EXAMPLE 5-B: PERFORMANCE ANALYSIS USING NON-LINEAR STATIC ANALYSIS (STATIC PUSHOVER ANALYSIS) OF BUILDING IN EXAMPLE -2 AFTER RETROFITTING

A.2.14. OUTPUT

The analysis is carried out for Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) as defined in IS 1893 (2002). The main results for both DBE and MCE are given below.

A.2.15. DESIGN BASIS EARTHQUAKE (DBE)

For Design Basis Earthquake (DBE), the performance point of retrofitted structure appears at Sa (spectral acceleration) equal to 0.382 and maximum roof deflection (D) is 0.525 inches as shown in the image below.

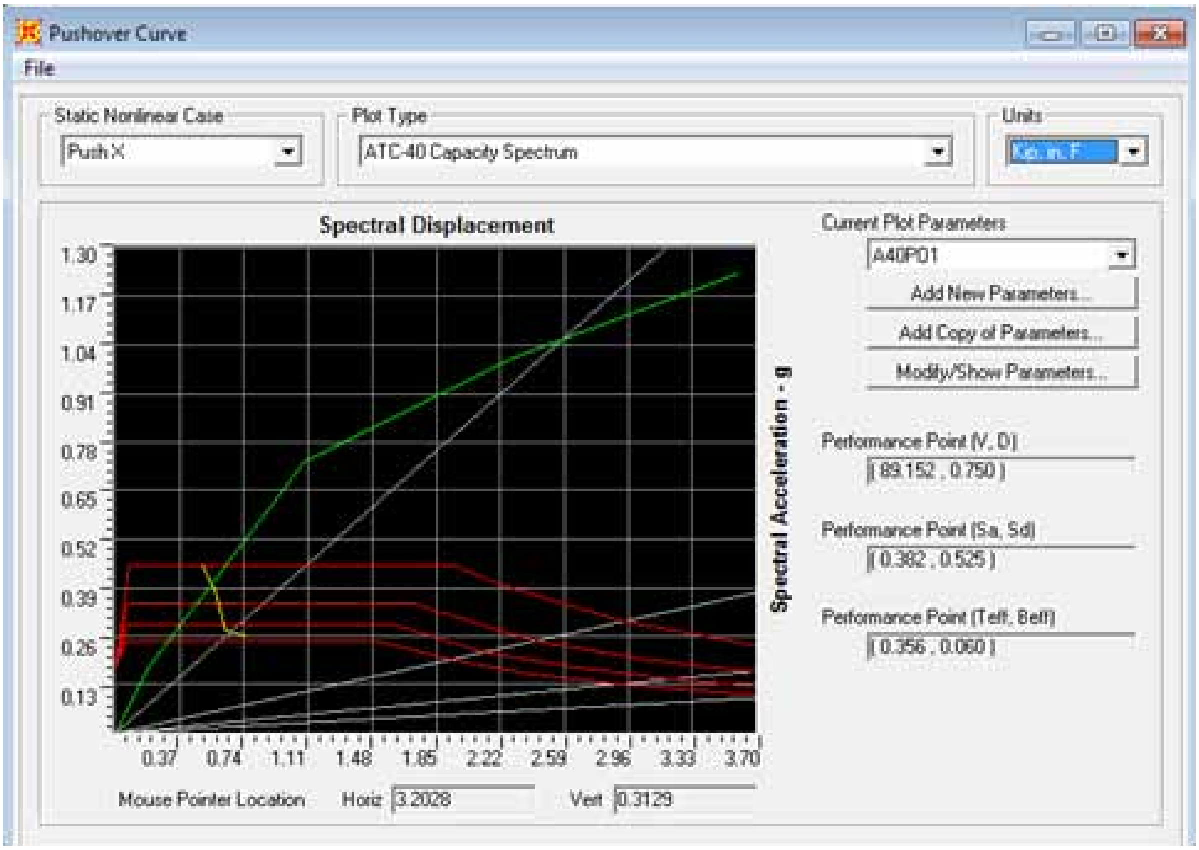


Figure A-17 Capacity Spectrum of Retrofitted Structure at DBE

A.2.16. Maximum Considered Earthquake (MCE)

For Maximum Considered Earthquake (MCE), the performance point is obtained at Base shear (V) and Roof Displacement (D) at 772.160 and 42.01mm, respectively. The spectral acceleration performance point is 0.744 which is higher than the spectral acceleration value of 0.36 given in IS 1893 codal provisions.

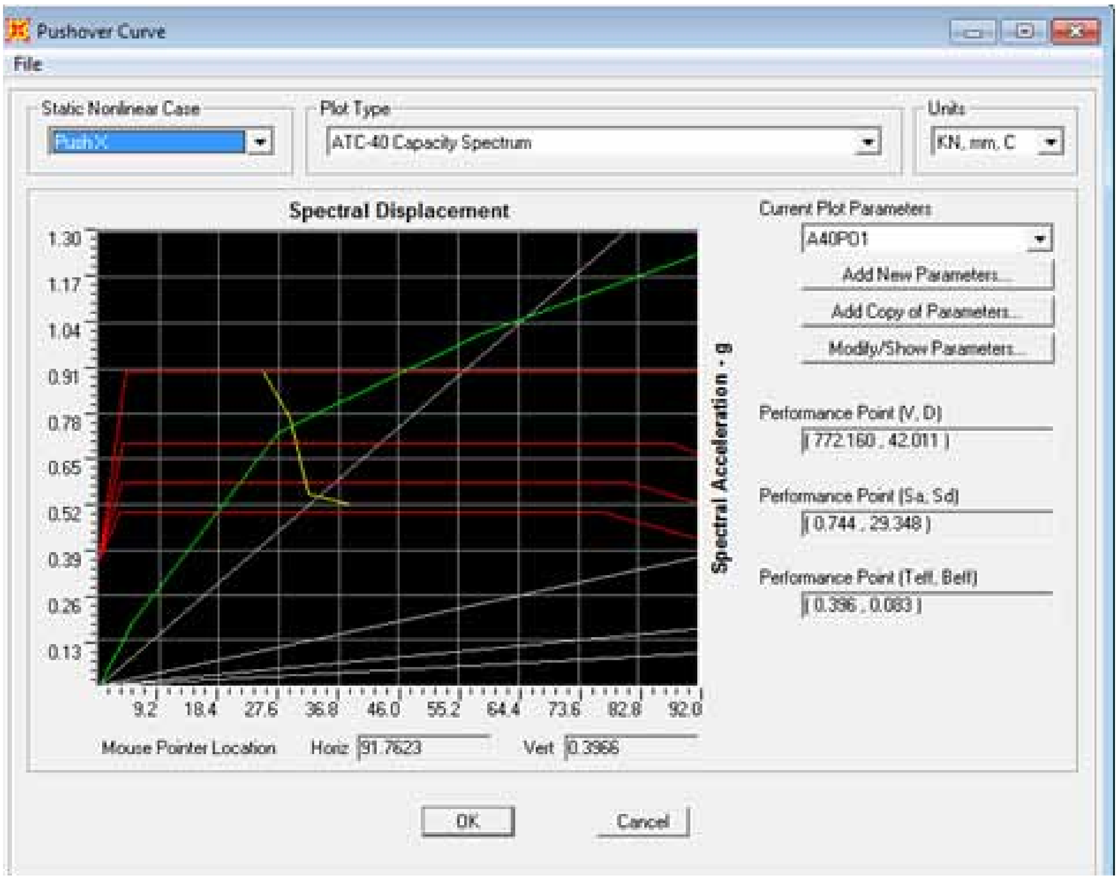
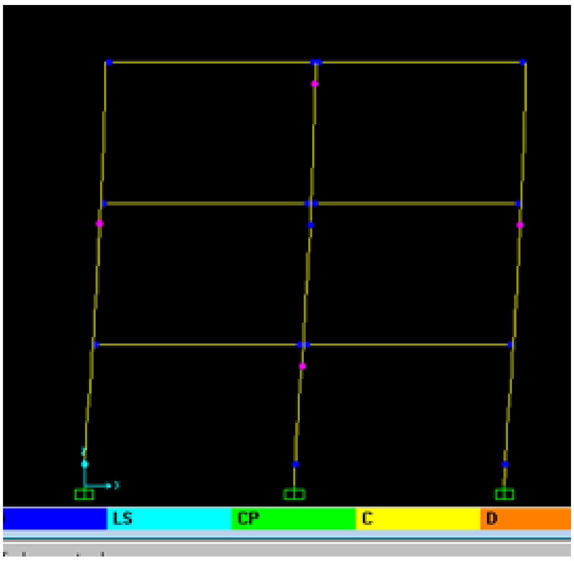


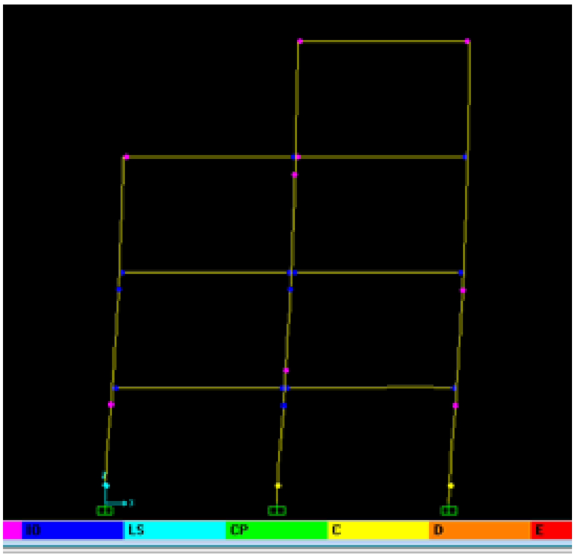
Figure A-18 Capacity Spectrum of Retrofit Structure at MCE

A.2.17. PLASTIC HINGES MECHANISM

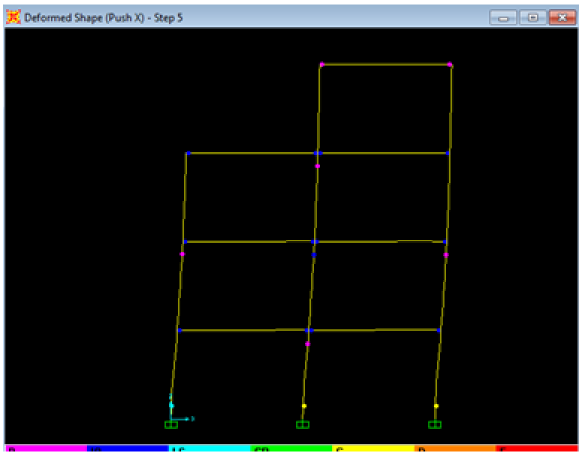
Plastic hinge formation for the building mechanisms have been obtained at different Displacement levels. The hinging patterns are as shown in figures:



Grid 1-1



Grid 2-2



Grid 3-3

A.2.18. CONCLUSION

Thus the analysis of the building after RC jacketing of columns was carried out by modeling the building in SAP 2000 and the results were found as mentioned above from the push over analysis. The building is found to be safer in Push over analysis.

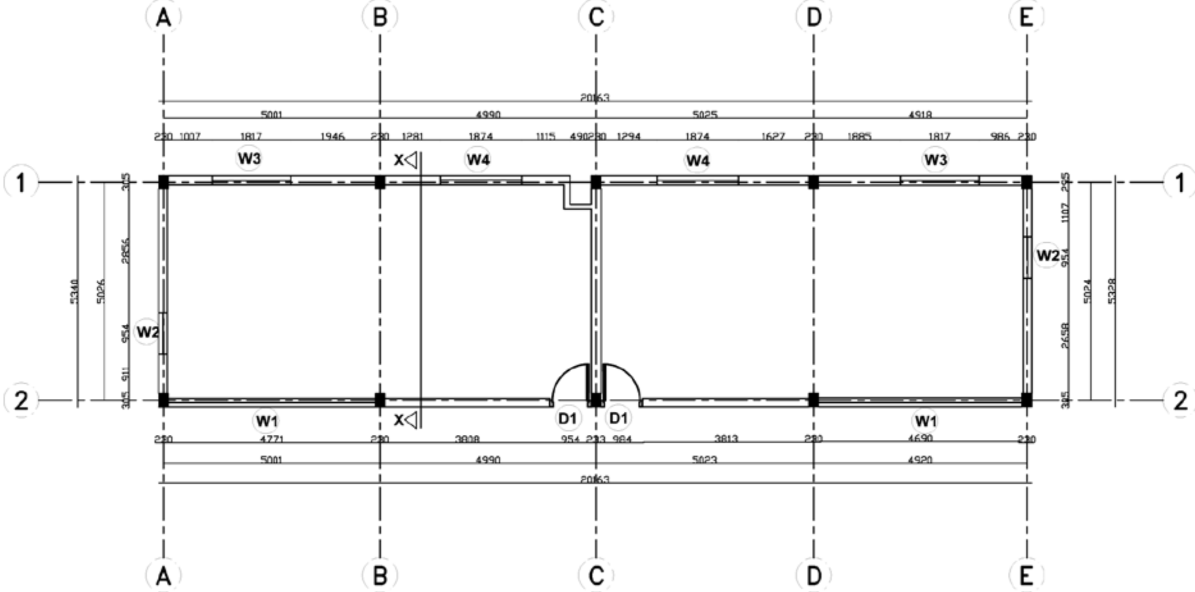
Hence the retrofitted building can be considered safe for the hospital purpose.

From the performance analysis of the retrofitted building according to the drawings provided and field verification, the retrofitted building will be in damage control level. The performance objective for DBE is expected at Life Safety level and for MCE it's expected at Structural stability. As the performance of the building is realized at damaged control level (better performance than the expected performance), **the building is safe to use as per the requirements of IS 1893-2002.**

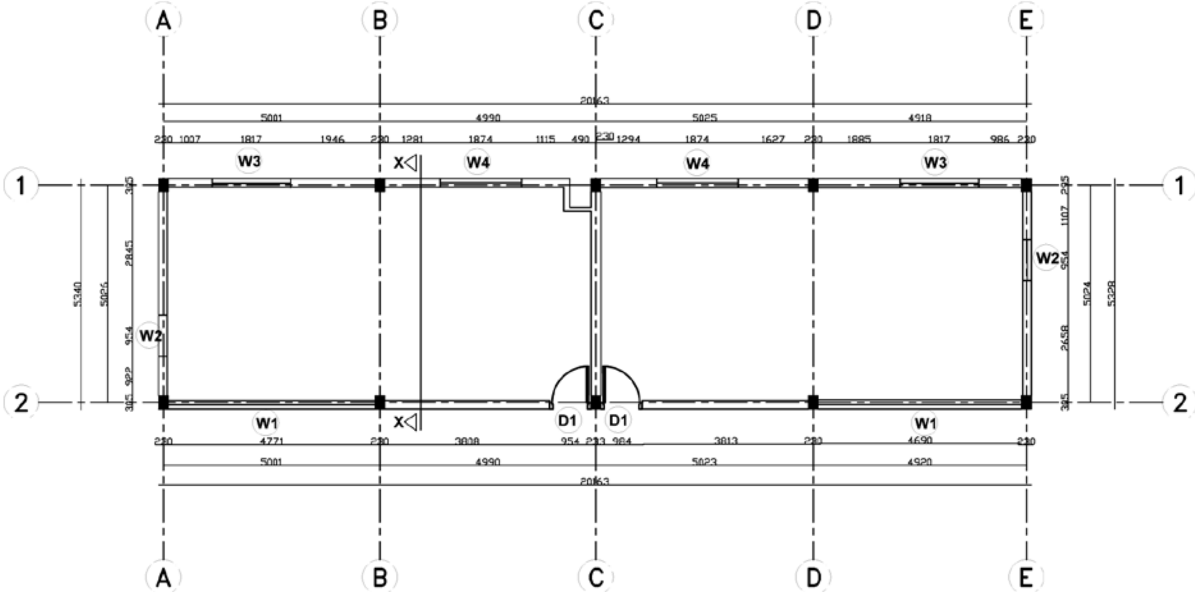
EXAMPLE 5-C: ANALYSIS AND RETROFITTING DESIGN OF SCHOOL BUILDING

A DETAIL EXAMPLE:

BUILDING DESCRIPTION



GROUND FLOOR PLAN



FIRST FLOOR PLAN

Figure A-19 Plan of the School Building

SUMMARY OF THE BUILDING

Shape of Building:	Rectangular
Building Dimension- Length (m):	19.949
-Width (m):	5.071
Plinth Area (sq.m):	101.16
Number of Storey:	2
Total Height of Building (m):	5.262
Building Type:	RC Framed Building

Wall Thickness (mm):	230
Joint Mortar:	Cement sand
Floor Finish :	Cement Punning
Type of Foundation:	RCC Isolated Foundation
Presence of- Lintel Band	No
-Sill Band	No

Structural Analysis & Retrofitting Design

The structural configuration of the building is Reinforced concrete 3-storied structure. The plan(Grid Lines) of the building is illustrated in Figure . The structural analysis of the building was done using linear static analysis with the help of structural analysis software named ETABS 2013 V 13.1.5.

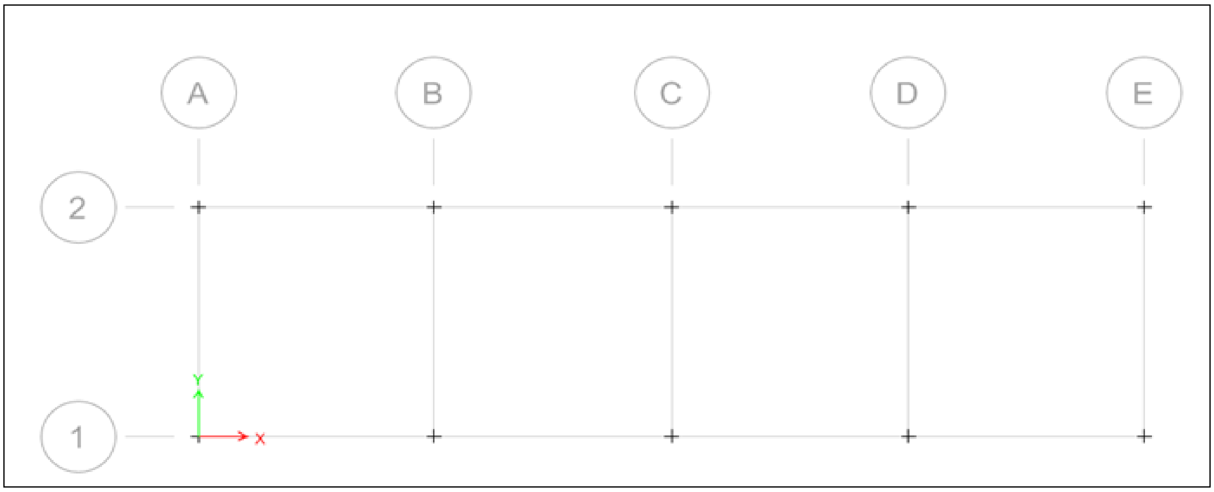


Figure A-20 Ground Floor Plan with Grid Lines of Analysis

GENERAL INPUT

Loads and Loading

The main types of loads considered for the design of building structure are vertical loads (dead and imposed load) and lateral load lateral loads (earthquake load).

a) Dead Loads:

The gravity loads due to self-weight of structural elements are determined considering the dimensions of elements and unit weights as per IS: 875 (part 1), 1987. The dead load is considered the weight of structural elements including walls, finishing work and all other permanent features in the building. The wall loads are calculated and applied on beams and slabs as per measured drawings

b) Live Loads:

The live load considered for various usage of space office, corroder, lobbies, parking and staircase are taken as per codal provision in IS: 875 (part 2), 1987.

c) Earthquake Loads:

Earthquake load is calculated using Seismic coefficient of equivalent static force analysis method for zone V according to the codal provision in IS: 1893 (part 1), 2002. The soil type is taken as III and Importance Factor is taken as 1.0. However only 70% of the lateral load is considered according to the cl. 5.4 of IS 15988: 2013 Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings - Guidelines

Types and Grades of Material

Concrete and reinforcement steel are main basic material for reinforced concrete structure. The concrete used in the building is of Grade M15, and steel is of Grade Fe 415.

Depth of foundation

The depth of foundation is mainly governed by factors such as scour depth and nature of subsoil strata to place foundation, basement requirement and other environmental factors. As there are no rivers in the immediate vicinity of the building site, chance of scouring is absent. The foundation depth is assumed as per general practices.

Structural Data

As per measured drawing and Non-Destructive tests carried out at the site following data were used for the structural analysis:

ID	Designation	Size(DXB)	Grade	Top Rebar	Bottom rebar
BEAM	B1	375x230	M20	3-20□	3-20□
ID	designation	size(DXB)	grade	rebar	
COLUMN	C1	230x350	M20	8-20□	
ID	designation	size(D)	grade	edge rebar	mid span rebar
SLAB	S-100		M15	X - 10 mmØ @ 100 mm C/C	X - 10mm Ø @ 100 mm C/C
				Y - 10mm Ø @ 100 mm C/C	Y - 10mm Ø @ 100 mm C/C

A.2.19 Modeling and structural Analysis

ETABS 2013 V 13was used as a tool for modeling and analysis of the building. ETABS 2013 V 13 is the most sophisticated and user friendly series of computer programs. Creation modification of models, execution of analysis and checking and optimization of the design can be done through this single interface. Structural analysis program ETABS 2013 V 13 is used for modeling and structural analysis to check the member capacity of structure as per existing design and construction.

Analysis Approaches for Building

The structure was modeled as a three dimensional ordinary RC moment resisting frame of main structural member beam and column to determine the required strength of the structure. The effect of infill brick wall is not considered while analyzing structure. The gravity (dead and live) load applied in combination with lateral load (seismic load) as recommended by IS 1893 (part1) 2002 in analysis. The analysis was performed for various combinations as per IS 1893 (part1):2002.

The analysis of the building was done based upon the following two cases:

1. Case I: Considering existing building
2. Case II: Considering existing building after improvement of structural members (Retrofitting)

Case I: Considering existing building

Analysis Input

TheFigure and Figure illustrate the 3d model and girds of the building for ETABS analysis respectively

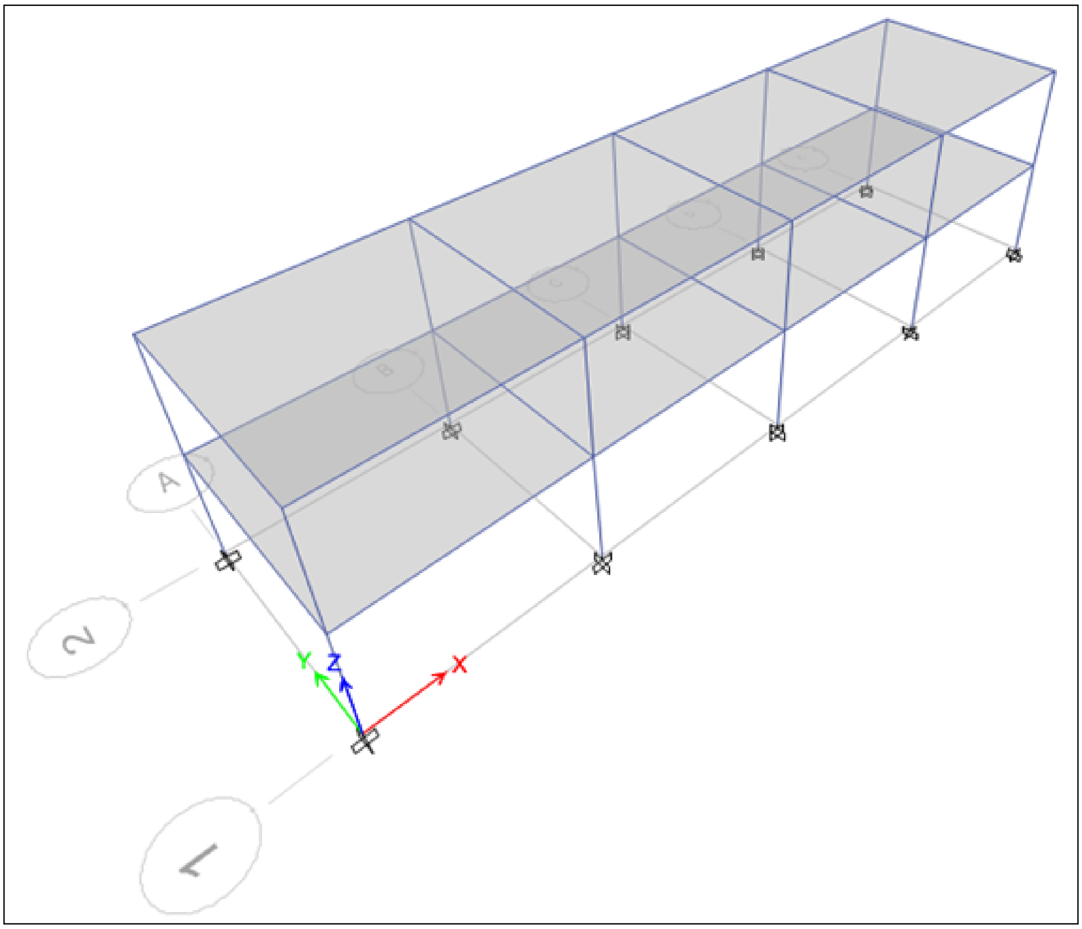


Figure A-21 3d model of the building with grid lines

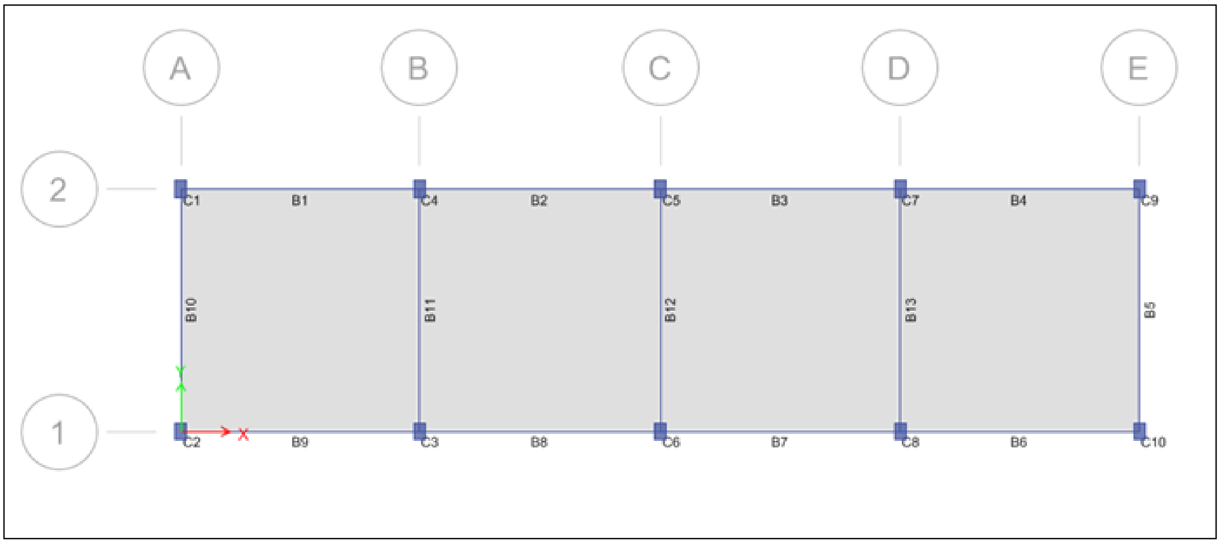


Figure A-22 Grid Lines with Beam and Column ID

The various parameters considered for analysis is presented below:

Zone factor,	Z	0.252
Importance Factor,	I	1.5
Reduction Factor	R	3
Damping	DAMP	0.05

Only 70% of the earthquake is considered here reducing zone factor to 0.252

Codal Calculation: As per IS 1893-(part1):2002

Clause 6.4.2: For any structure with T<0.1, Ah shall not be less than Z/2

Cl.7.6.2, Tx= $0.09h/\sqrt{Dx}$

h	= 5.262	m	Dx	=	19.49	m
Dy	= 5.07	m	Tx	=	0.1073	Sec
Ty	= 0.2103	Sec				

The building was analyzed as per fore-mentioned criteria and findings are shown below. The existing building was found to be safe in drift criteria. Also, given below is the table which compares existing reinforcements with the required values from analysis:

Figure A-23 Grid 1-1

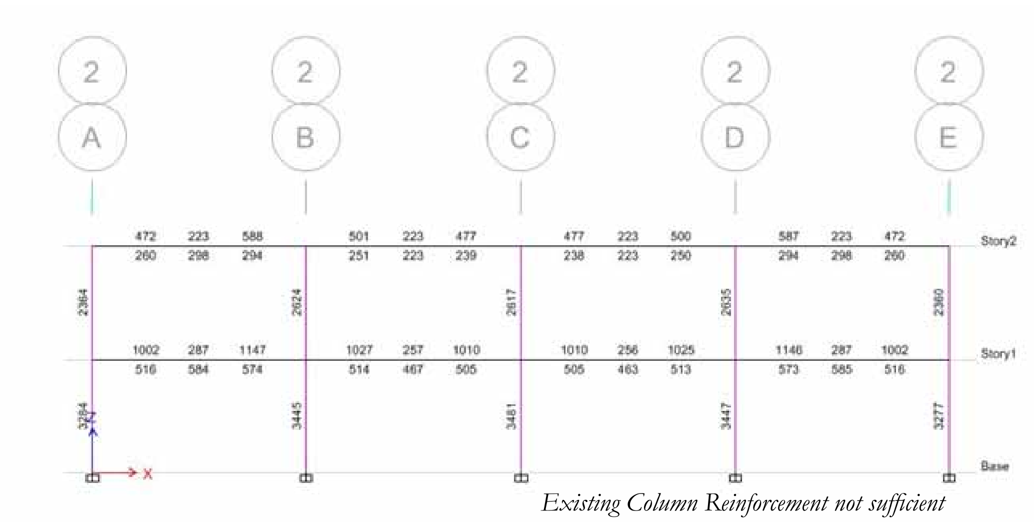


Figure A-24 Grid 2-2

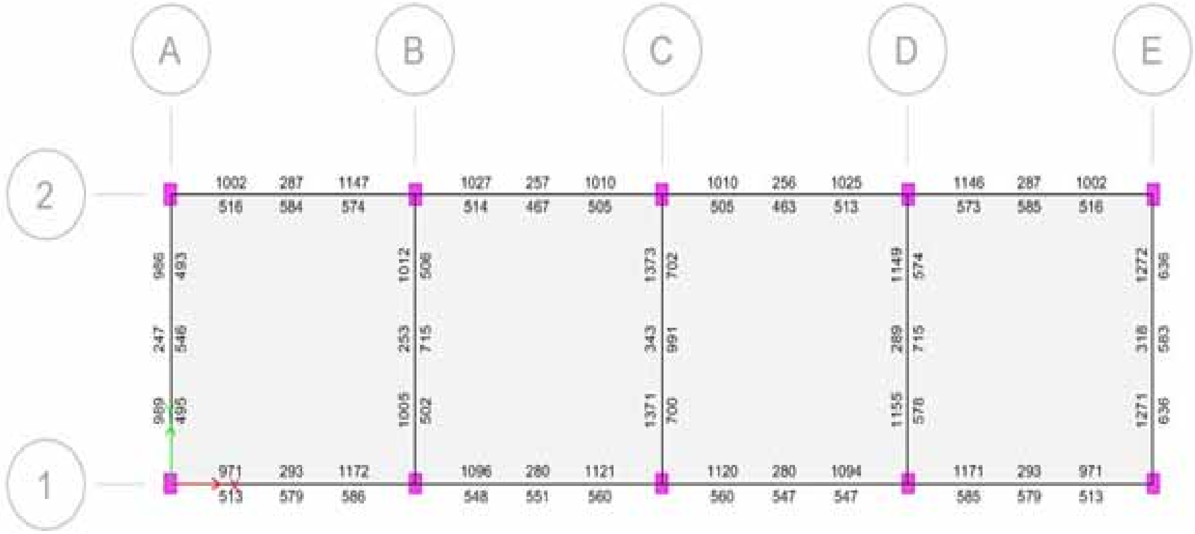


Figure A-25 Storey 1

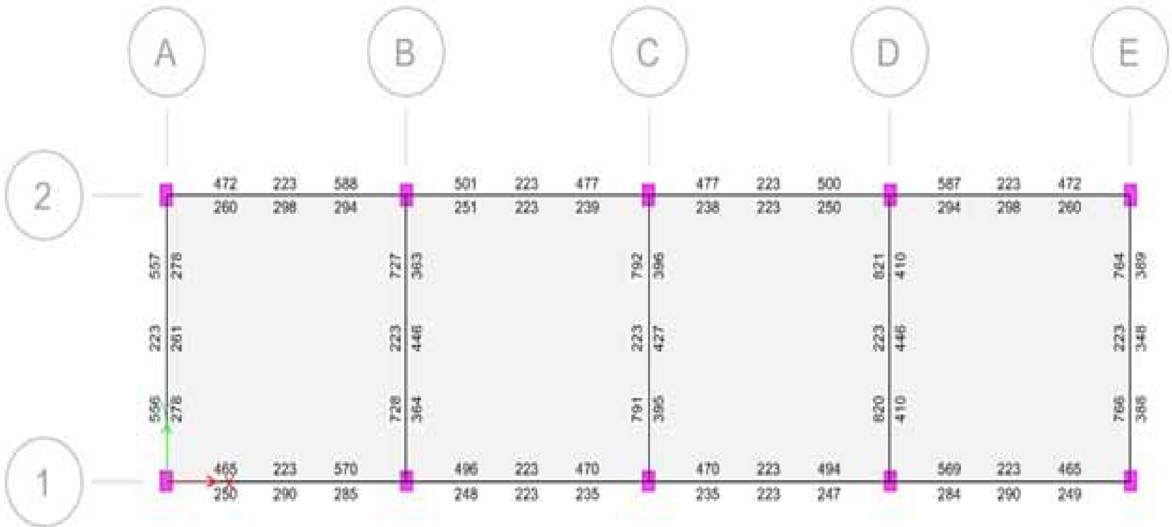


Figure A-26 Storey 2

It is observed that the size of the beam and columns is not appropriate for the existing structure. Also, the required reinforcements are seen to be on the higher side than the existing reinforcement area and thus the building requires some seismic strengthening as the overall condition of the building seemed to be vulnerable to earthquakes. The details of existing and required reinforcements for the beam is shown in Error! Reference source not found.whereas the details of existing and required reinforcements in columns in shown in Table 1.

Table1: Required and existing reinforcements in beams

TABLE: Concrete Beam Flexure Envelope			Indian code			
Label	Story	Section	As Top mm²	As Bot mm²	Area Existing Top	Area Existing Bottom
B6	Story2	B-230X375-M20	569	290	942	942
B7	Story2	B-230X375-M20	494	247	942	942
B8	Story2	B-230X375-M20	496	248	942	942
B9	Story2	B-230X375-M20	570	290	942	942
B5	Story1	B-230X375-M20	1272	636	942	942
B6	Story1	B-230X375-M20	1171	585	942	942
B7	Story1	B-230X375-M20	1120	560	942	942
B8	Story1	B-230X375-M20	1121	560	942	942
B9	Story1	B-230X375-M20	1172	586	942	942

Beam reinforcement lagging

Table 2: Required and existing reinforcements in columns

Label	Story	Section	P	M Major	M Minor	Required steel(IS)	Main Bar Existing	
			kN	kN-m	kN-m	sq. mm	area	%
C2	Story2	C-230*350-M20	76.6208	-41.7207	-29.3236	2371	2512	3.12%
C3	Story2	C-230*350-M20	111.6689	-42.5493	-28.5133	2384	2512	3.12%
C6	Story2	C-230*350-M20	108.9562	39.4353	-41.3975	2514	2512	3.12%
C8	Story2	C-230*350-M20	111.5674	42.4764	-28.5031	2380	2512	3.12%
C10	Story2	C-230*350-M20	76.6452	41.7427	-29.3268	2372	2512	3.12%
C2	Story1	C-230*350-M20	219.6013	-63.6746	-14.0951	3400	2512	3.12%
C3	Story1	C-230*350-M20	293.2828	-64.6745	-9.1219	3350	2512	3.12%
C6	Story1	C-230*350-M20	350.7592	63.1464	-14.7266	3441	2512	3.12%
C8	Story1	C-230*350-M20	292.9997	64.6451	-9.1174	3348	2512	3.12%
C10	Story1	C-230*350-M20	219.6563	63.6819	-14.0897	3400	2512	3.12%

Case II: Considering existing building after improvement of structural members (Retrofitting)

The results in the case I show that the building is not safe in existing conditions with regard to the lateral load considered and thus the building has to be retrofitted. This case will see the possibility of increment in the size of columns. The reinforcements in the existing building also seem to be on lesser side. Thus, nominal increase in the reinforcements are done while increasing the sizes of the frames whose reinforcements are found to be okay with the existing ones.

Analysis Input
The Figure A-27 illustrates the 3d model and girds of the building for ETABS analysis.

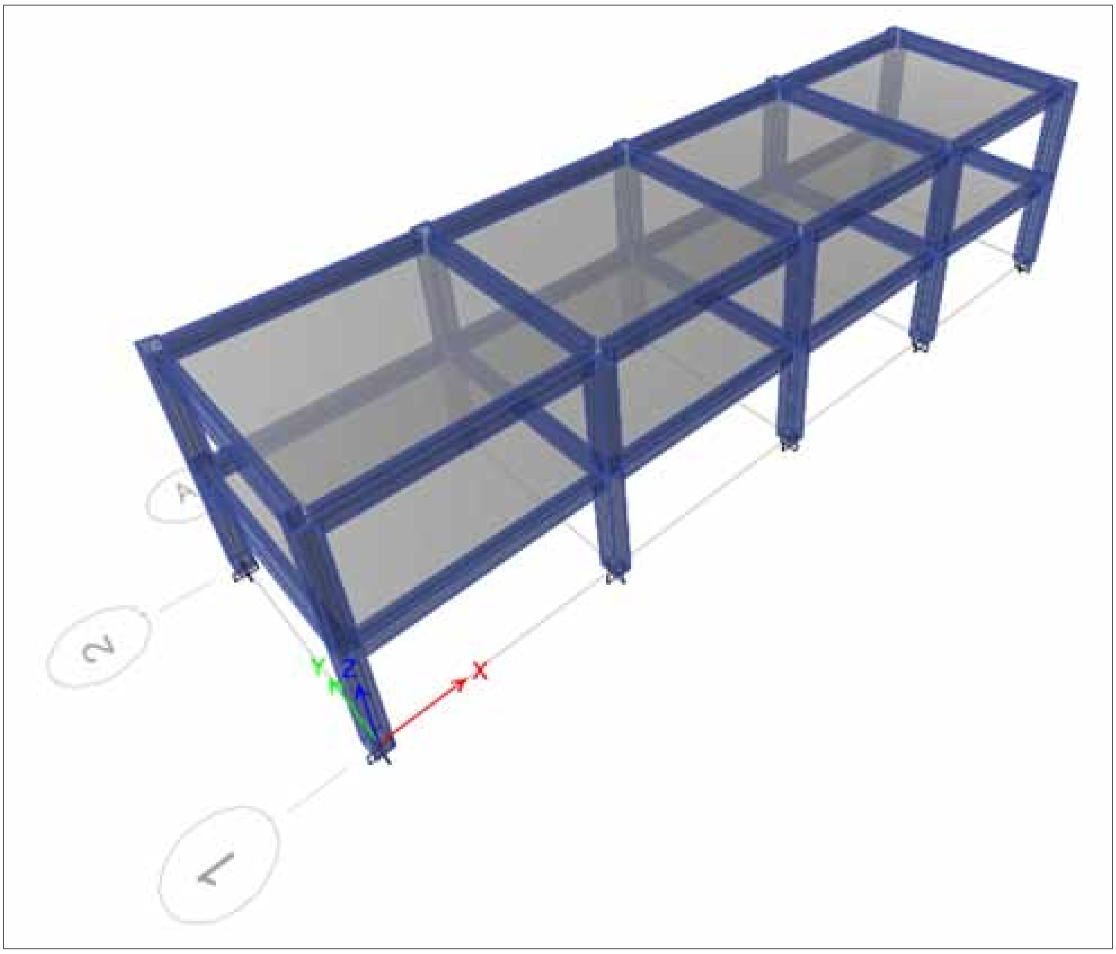


Figure A-27 3d model of the building with grid lines

The size of the structural members recommended after retrofitting is given below:

ID	Designation	size (DXB)	Grade	Top Rebar	Bottom Rebar
BEAM	B1	375x230	M20	3-20□	3-20□
ID	designation	size(DXB)	grade	rebar	
COLUMN	C1	430x550	M20	8-20□	
ID	designation	size(D)	grade	edge rebar	mid span rebar
SLAB	S-100		M15	X - 10 mmØ @ 100 mm C/C	X - 10mm Ø @ 100 mm C/C
				Y - 10mm Ø @ 100 mm C/C	Y - 10mm Ø @ 100 mm C/C

The beams and columns ID for the floor are shown in Figure 20:

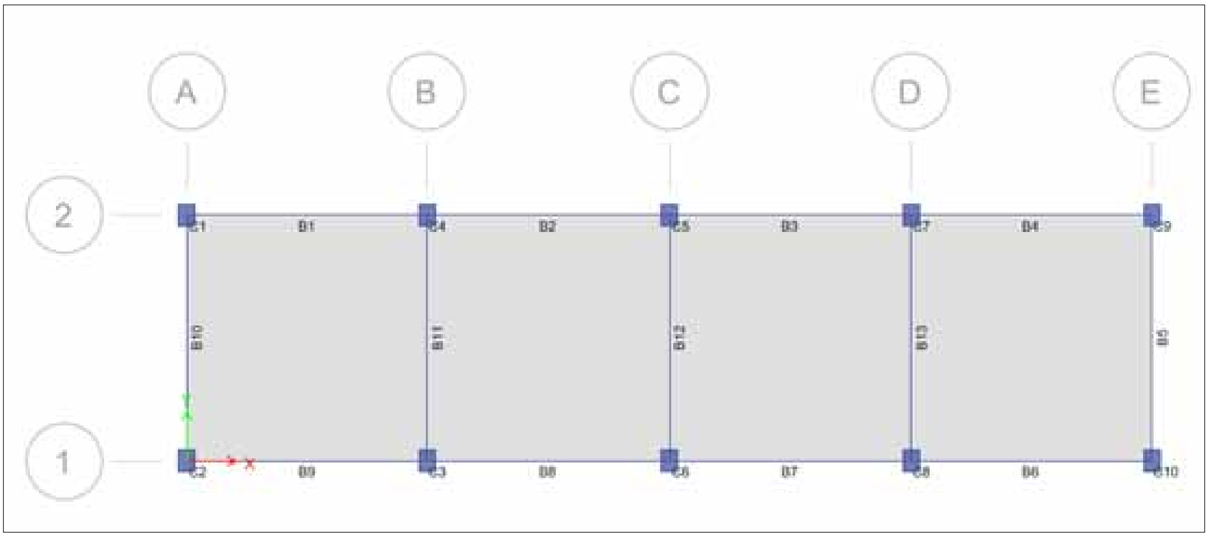


Figure A-28 Grid Lines with Beam and column ID

The details of existing and required reinforcements for the beam are shown in Table , whereas the details of existing and required reinforcements in columns in shown in

Table 3: Required and existing reinforcements in beams

TABLE: Concrete Beam Flexure Envelope							Area Provided	
Label	Story	Section	As Top mm^2	As Bot mm^2	Area Existing			
					top	bottom	top bar mm^2	bottom bar mm^2
B6	Story2	B-230X375-M20	567	283	942	942	Not Required	Not required
B7	Story2	B-230X375-M20	527	263	942	942	Not Required	Not required
B8	Story2	B-230X375-M20	528	264	942	942	Not Required	Not required
B9	Story2	B-230X375-M20	567	284	942	942	Not Required	Not required
B6	Story1	B-230X375-M20	851	514	942	942	Not Required	Not required
B7	Story1	B-230X375-M20	902	612	942	942	Not Required	Not required
B8	Story1	B-230X375-M20	904	617	942	942	Not Required	Not required
B9	Story1	B-230X375-M20	851	514	942	942	Not Required	Not required

Table 4: Required and existing reinforcements in columns

Label	Story	Section	P	M Major	M Minor	Required steel(IS)		main bar existing	Reinforcement to be added(As')
			kN	kN-mm	kN-mm	%	sq. mm	area	
C2	Story2	C-430*550-M20	79.2834	-29.4869	-34.7113	0.80%	1188	2512	Not required
C3	Story2	C-430*550-M20	132.834	-2.881	-41.9717	0.80%	1188	2512	Not required
C6	Story2	C-430*550-M20	130.842	2.6168	-57.0385	0.80%	1188	2512	Not required
C8	Story2	C-430*550-M20	132.687	2.7588	-41.9376	0.80%	1188	2512	Not required
C10	Story2	C-430*550-M20	93.5105	22.2236	-66.0542	0.80%	1193	2512	Not required
C2	Story1	C-430*550-M20	152.175	-81.0547	-11.7887	1.24%	1839	2512	Not required
C3	Story1	C-430*550-M20	182.102	-83.7444	-6.5989	1.21%	1802	2512	Not required
C6	Story1	C-430*550-M20	219.029	82.4681	-8.6342	1.12%	1665	2512	Not required
C8	Story1	C-430*550-M20	182.017	83.733	-6.5953	1.21%	1802	2512	Not required
C10	Story1	C-430*550-M20	161.564	3.2313	-128.5347	1.33%	1982	2512	Not required

Note: The amount of concrete and steel according to IS 15988:2013 Cl.8.5.1.1 to account for losses:

Ac' = (3/2) Ac and As' = (4/3) As

Where,
Ac and As = actual concrete and steel resp. to be provided in the jacket.
Ac' and As' = resp. concrete and steel values obtained for the jacket after deducting the existing concrete and steel from their respective required amount.

Hence minimum reinforcement (4-16 dia+ 4-12 dia) is added to the increased section of the column.

Similarly, spacing of ties to be provided in the jacket in order to avoid flexural shear failure of column and provide adequate confinement to the longitudinal steel along the jacket is given as:

$$s = (f_y/\sqrt{f_{ck}})*(d_h^2/t_j)$$

Where,
f_y = yield strength of steel
f_y = cube strength of concrete
d_h = diameter of stirrups
t_j = thickness of jacket.

A.2.20. Retrofitting Drawing

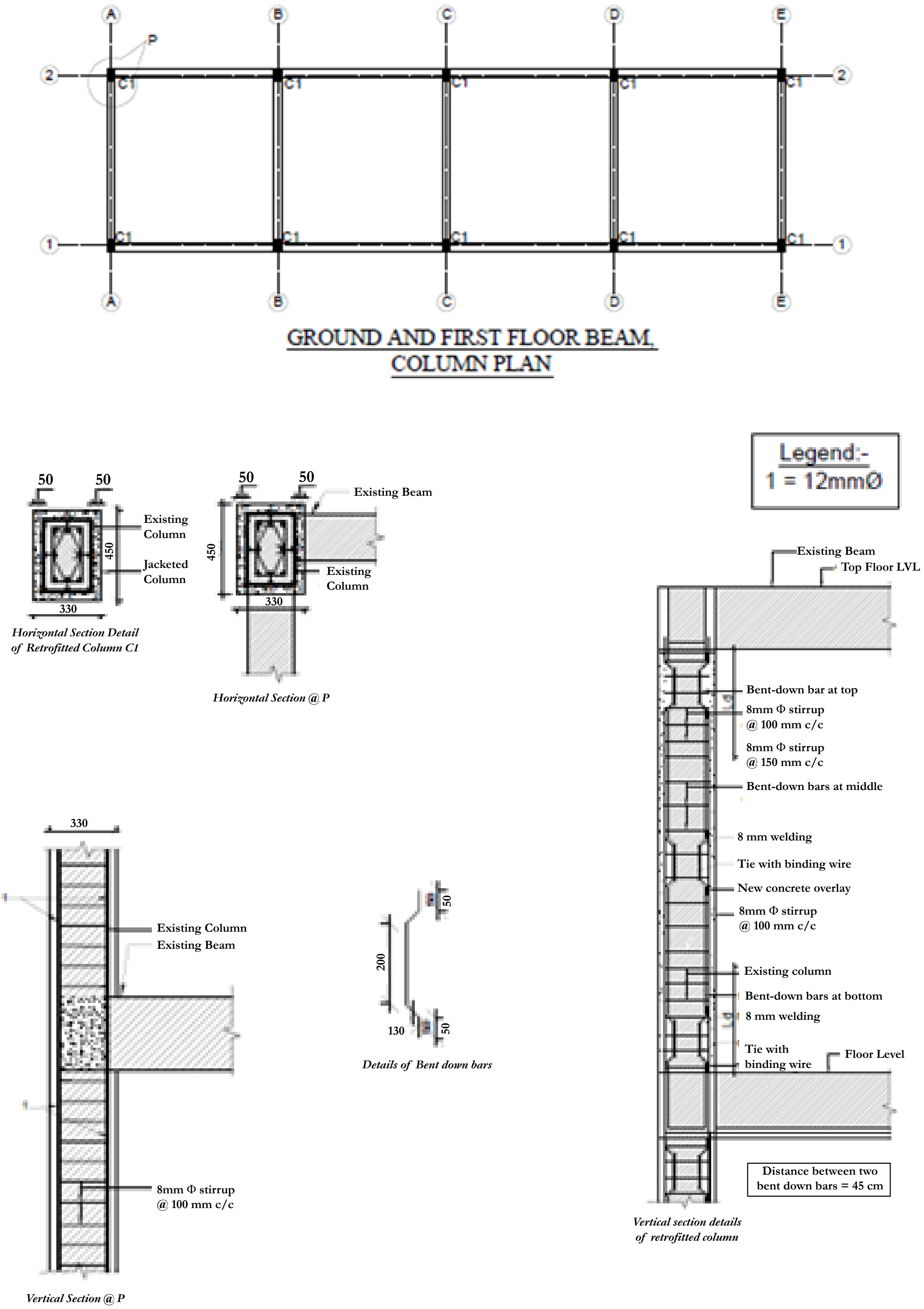


Figure A-29 Showing Column Jacketing

ANNEX B: CASE STUDY

B.1 ANALYSIS AND RETROFITTING DESIGN OF SDN 13 SYAMTALIRA ARUN

B.1.1 BUILDINGDESCRIPTION

B.1.1.1 INTRODUCTION

This is a project under Save the Children project, with design and technical assistance from Syiah Kuala University. SDN 13 SyamtaliraArun is located at North Aceh. The school building consists of 2 rooms. The school has approximately 400 students. In general, the structural system before retrofitted was reinforced concrete frames with infill masonry walls.

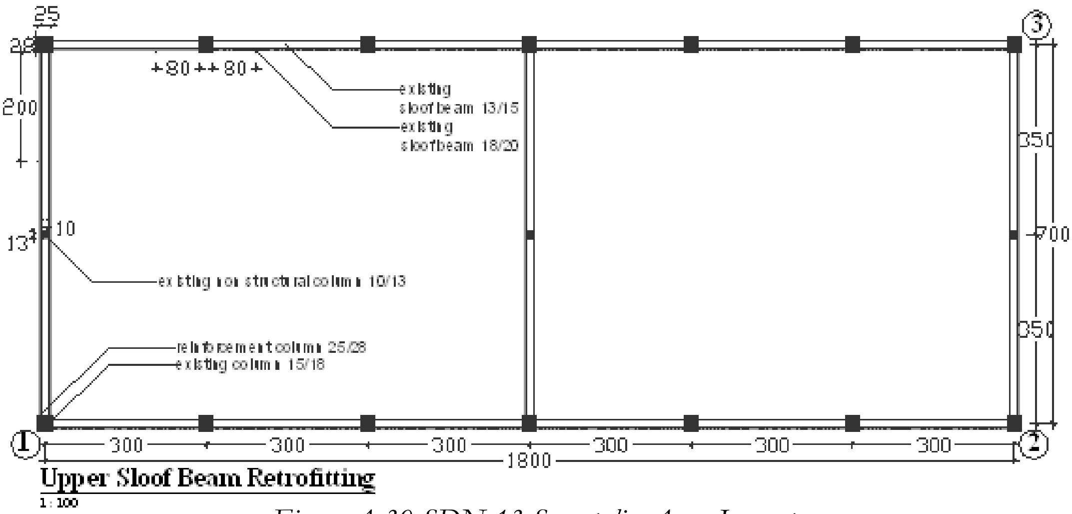


Figure A-30 SDN 13 SyamtaliraArun Layout

Figure A-31 Existing Condition of SDN 13 SyamtaliraArun

B.1.1.2 BACKGROUND

a. *Status and Condition of Structure*

From the initial survey, there were some major problems found in SDN 13 SyamtaliraArun, i.e:

1. Cracks on walls
2. Cracks on structural member
3. Poor workmanship
4. Poor quality construction

B.1.2 VULNERABILITY ASSESSMENT

(cracks observed, sizes of elements/ walls and others)

a. Assessment

1) **Visual Assessment**

In the visual assessment, the following measures were conducted:

- Rapid visual inspection and assessment
- Collection of design and drawing
- Topographical information of site
- Site measurement of main structural member
- Inspection of cracks and location
- Judgment of the construction quality
- Evaluation of workmanship
- Inspection of material used and its quality

Figure A-32 Visual Assessment

2) **Technical Assessment**

Based on the results from the visual assessment, the technical assessment was conducted. In the technical assessment, some of the physical verification and partial/non-destructive tests were carried out, and the technical assessment measures included:

- Review and evaluation of design, specification & drawing
- Comparison of size, quality between design drawing and state of the structure in site
- Check with code provision, mainly size of main structural member, reinforcement bar

3) **Results**

Based on the assessment, the following problems were found:

- Defect on the design
- Not satisfied code requirement
- Not satisfied new Code requirement (new provision after tsunami)
- Insufficient size of Structural member
- Improper site for foundation in some case
- Poor quality of material - Not satisfied Specification
- Poor workmanship

Source: Photos by Hari. D. Shrestha Other than stated

Figure A-33 Technical Assessment

B.1.3. RETROFITTING DESIGN

a. Design Recommendation

Retrofitting strategy was decided based on the results of technical assessment. Due to the approach of open frame system (walls were not considered as lateral resisting elements), the retrofitting design required that structural element sizes (beams and columns) to be increased to provide larger load resistance capacity. Hence, the following design approaches were proposed:

1) **Retrofitting on structural member**

FOUNDATION

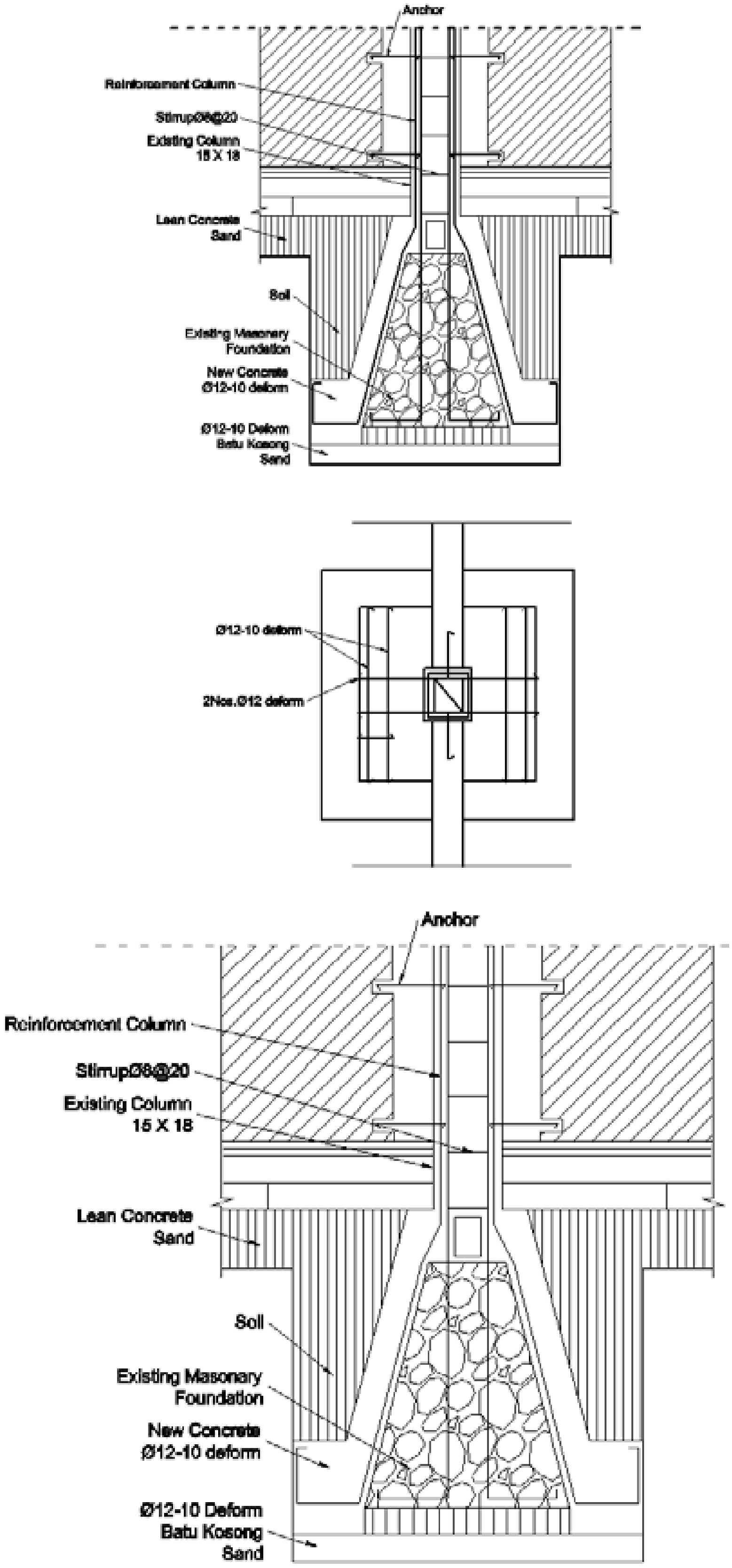
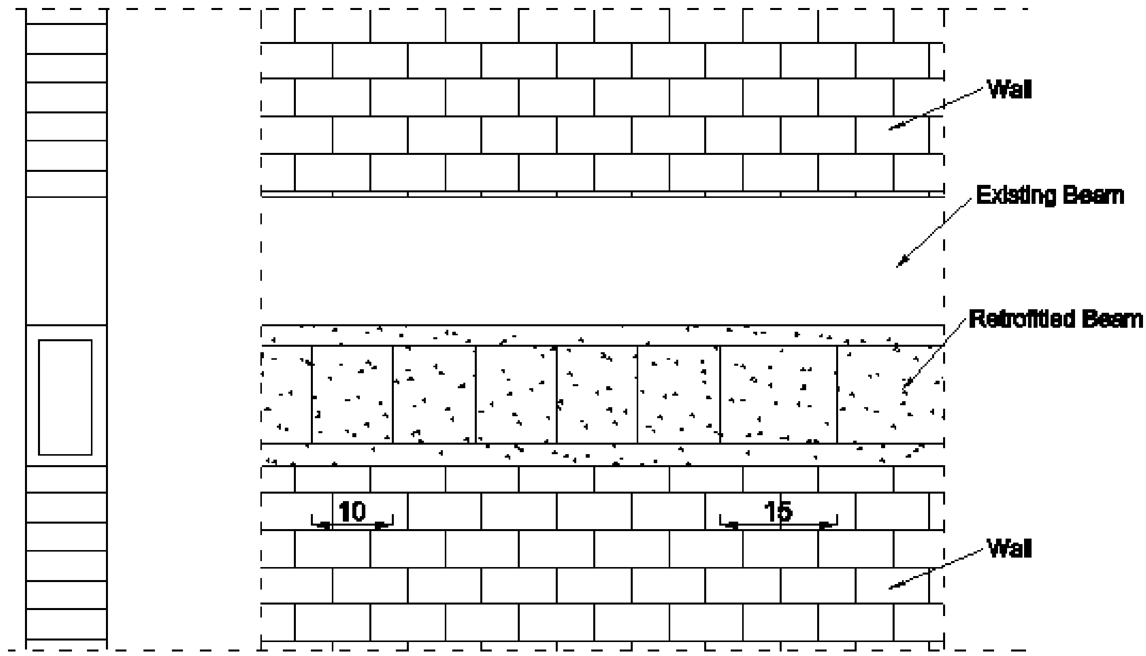


Figure A-34 Retrofitting of Foundation



Beam Retrofitting

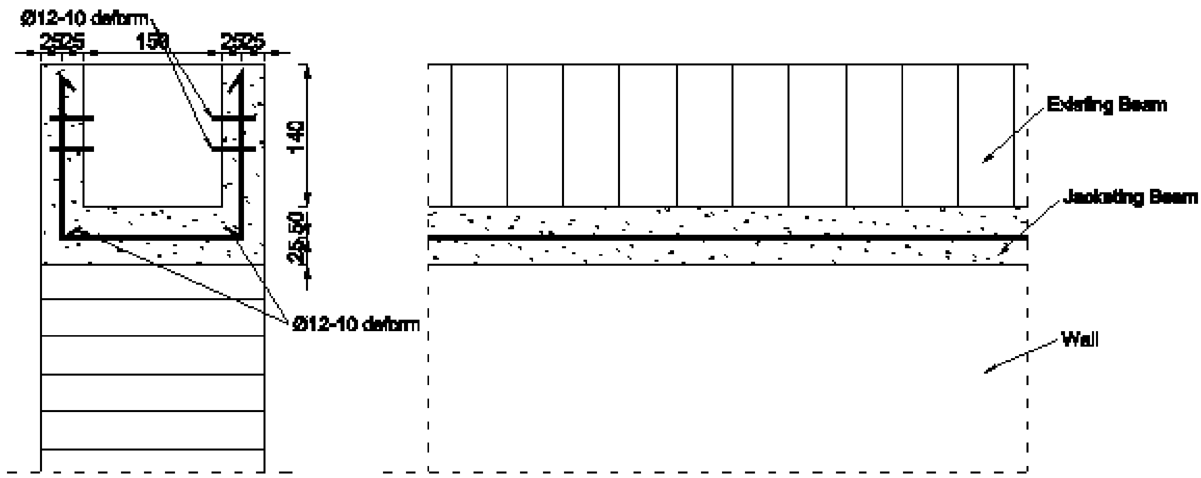


Figure A-35 Retrofitting of Beam

COLUMN

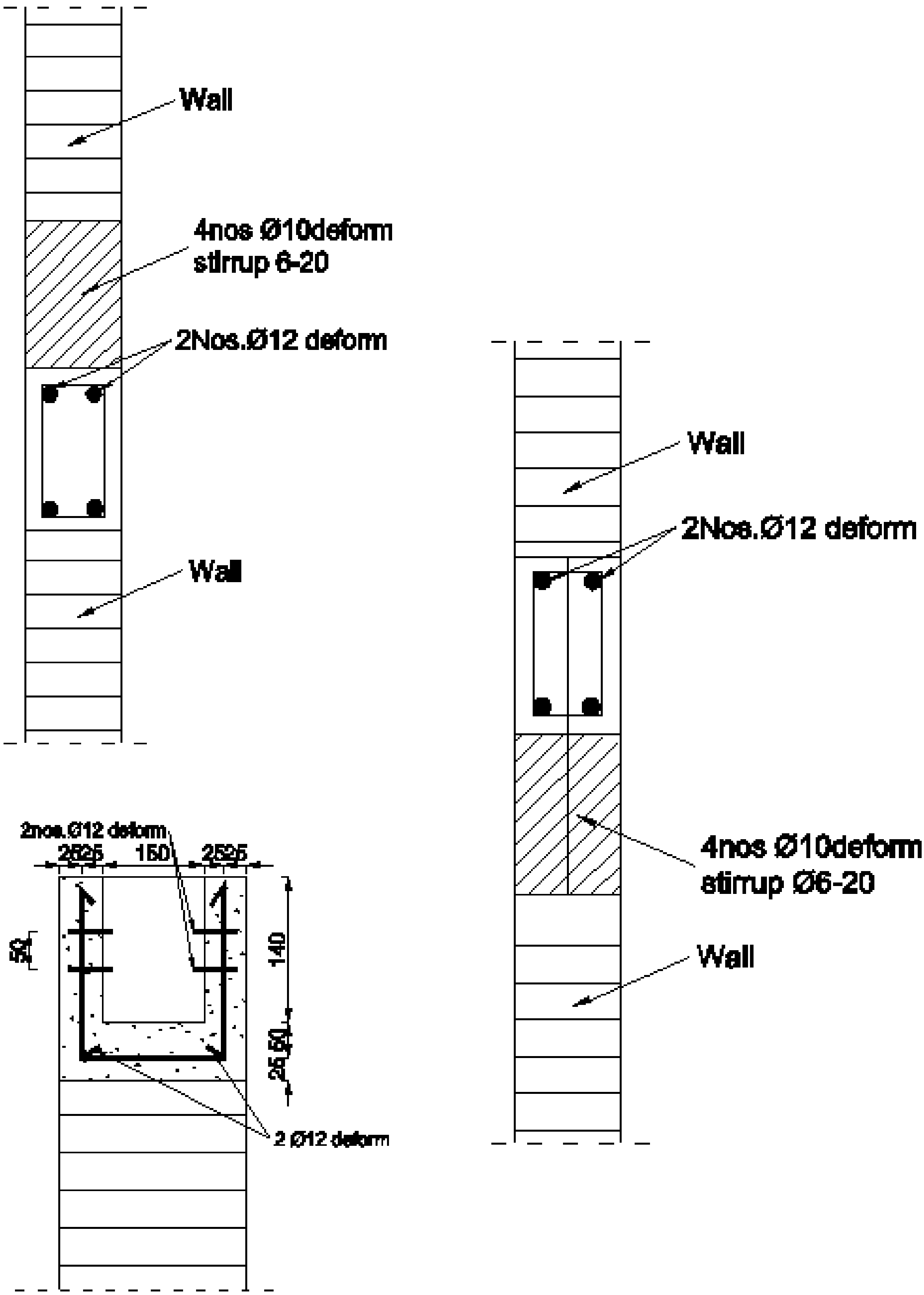


Figure A-36 Retrofitting of Beam Figure

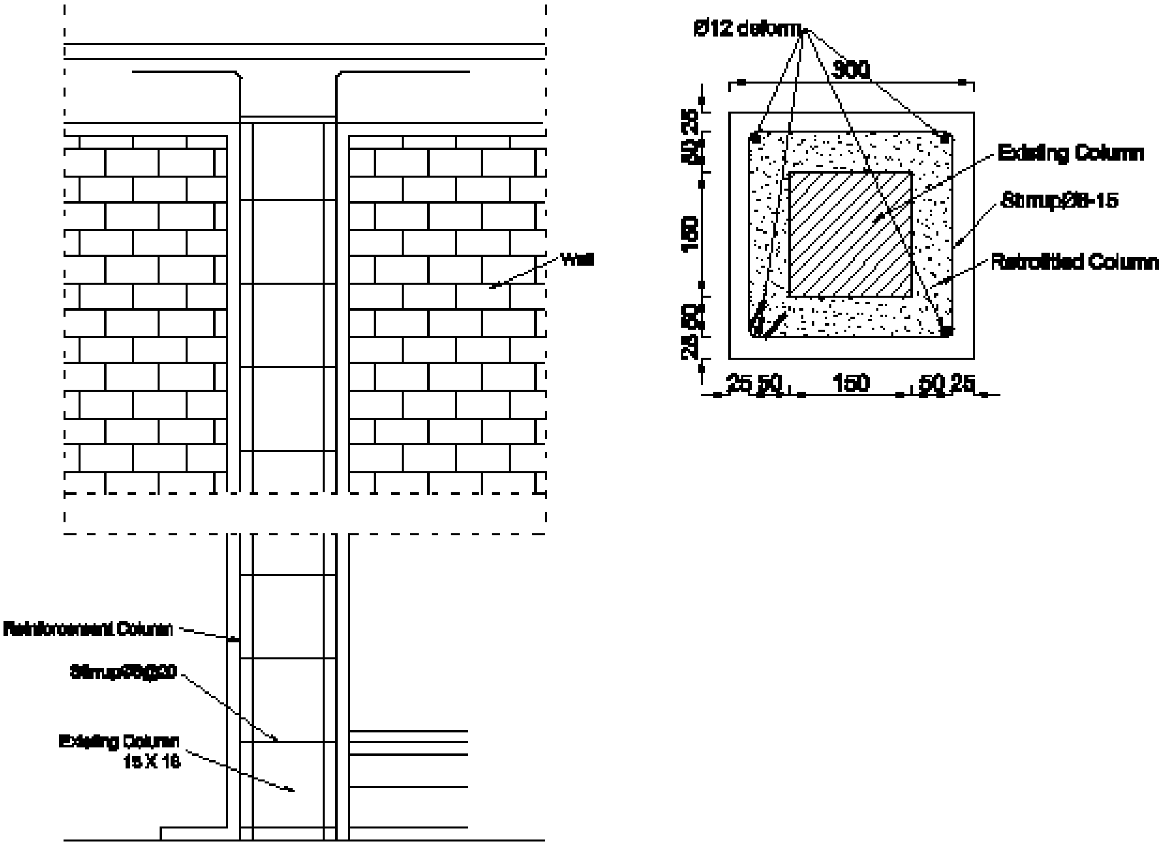


Figure A-37 Retrofitting of Column

2) **Connection between Wall & Column**

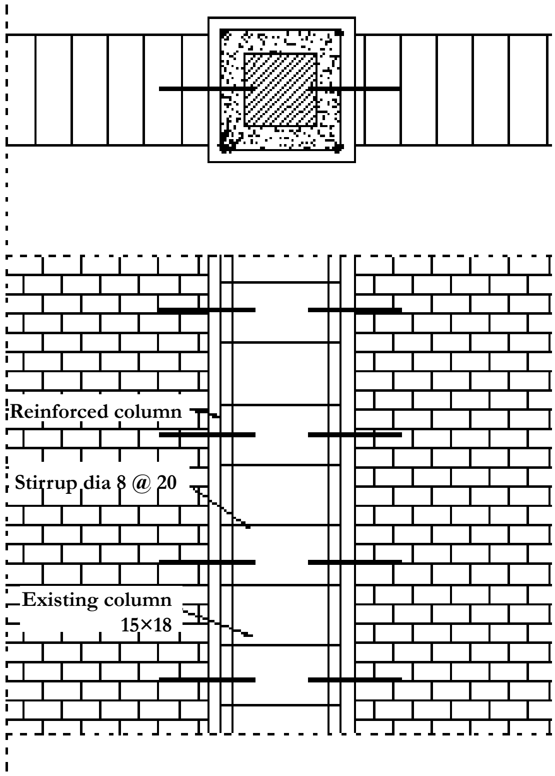


Figure A-38 Retrofitting between Wall and Column

3) **Retaining structures to protect Foundation**

4) **Corrective measure on cracks**

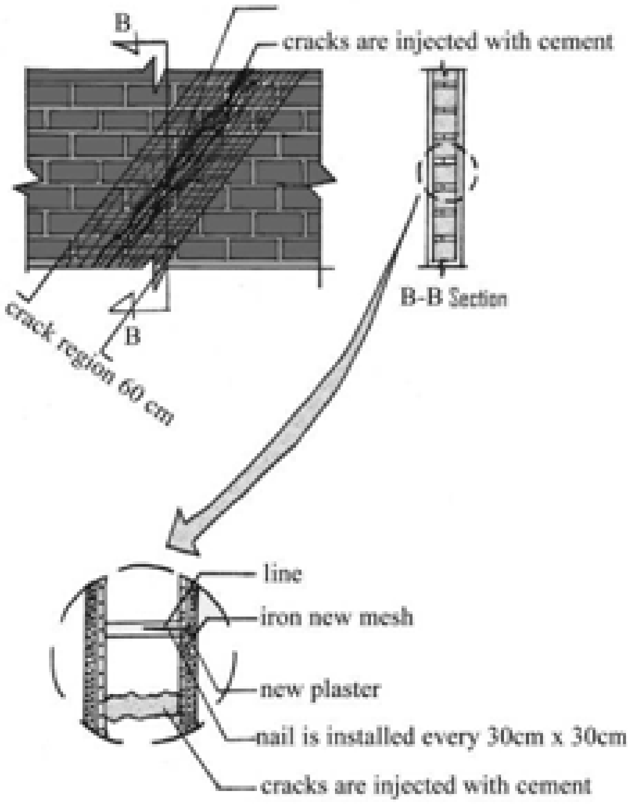


Figure A-39 Cracks Injection

b. **Retrofitting Process**

Figure A-40 Retrofitting of Foundation

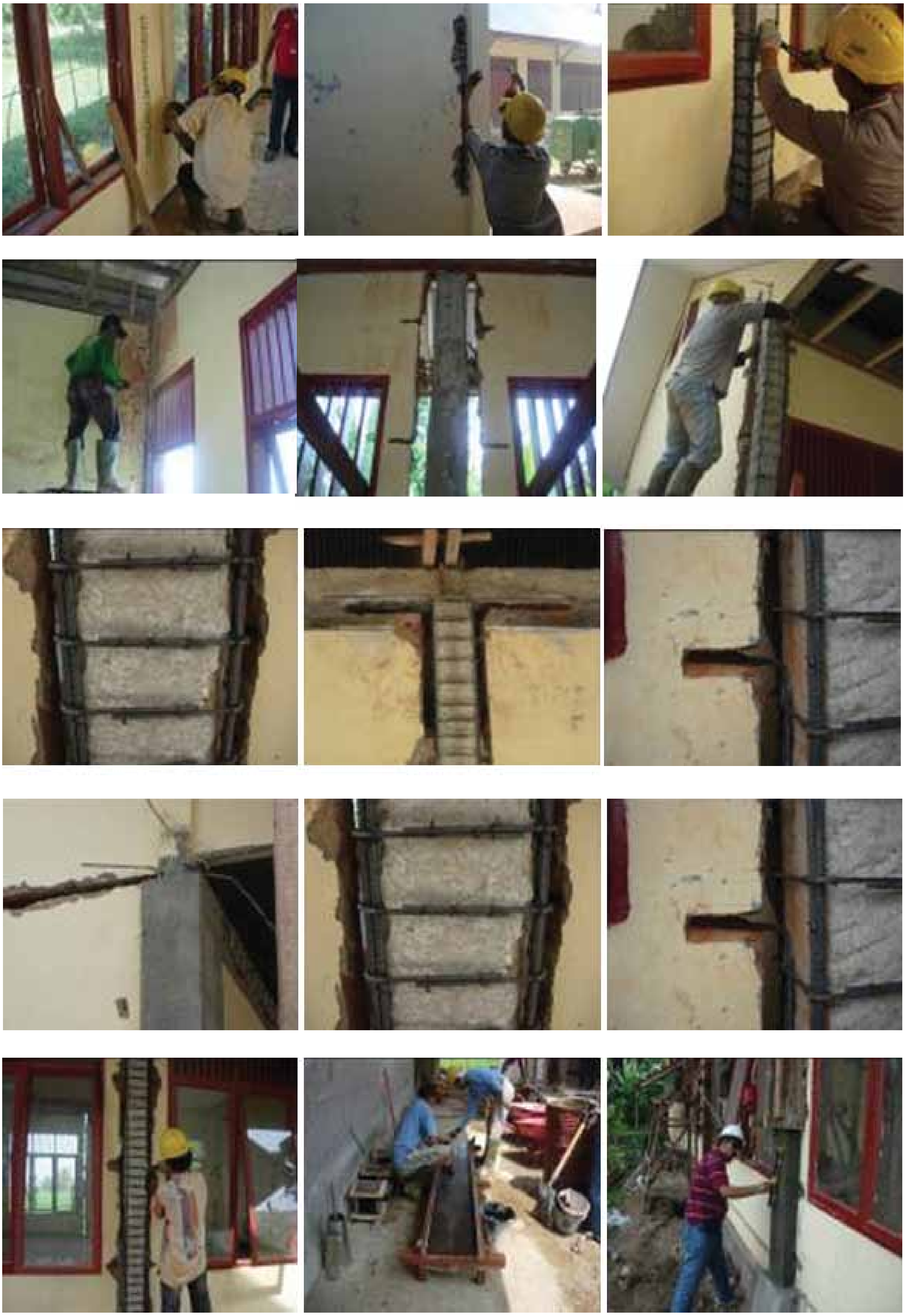


Figure A-41 Column Retrofitting

Figure A-42 Beam Retrofitting

B.1.4. IMPLEMENTATION

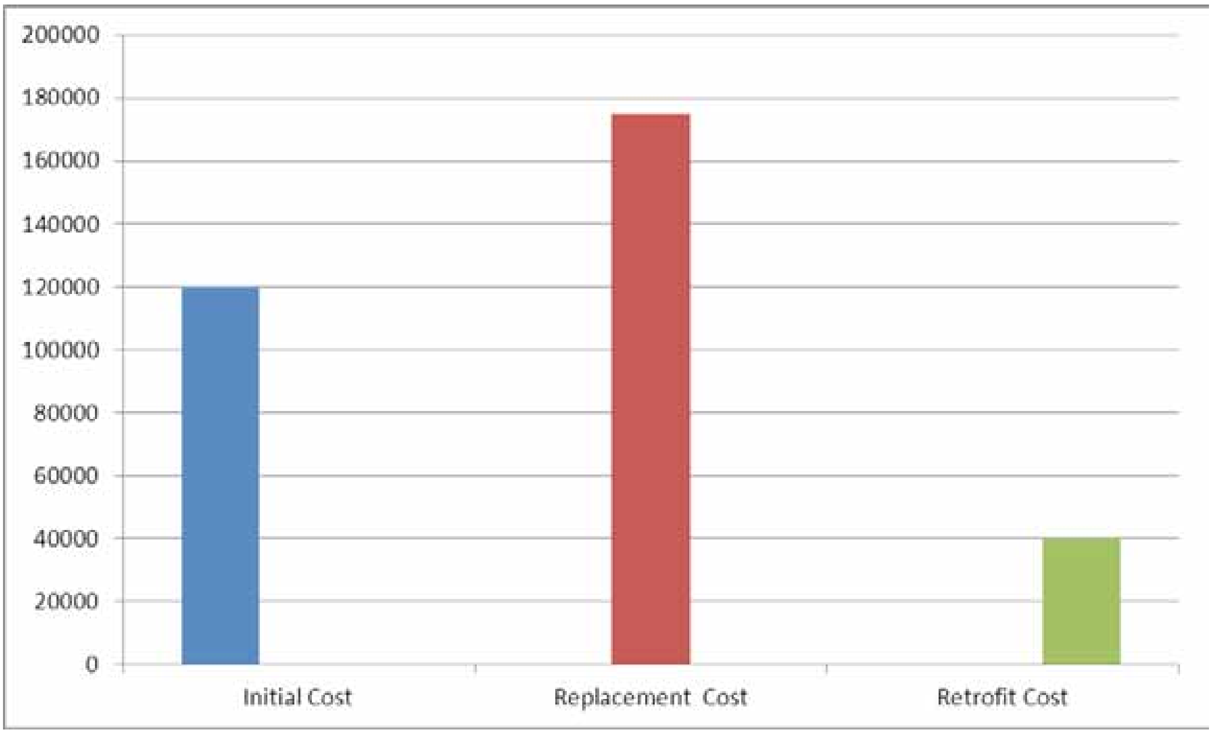
Figure A-43 Retrofitted Structure

B.1.5. COST CALCULATION

Initial Construction Cost : US$ 120,000

Replacement Cost : US$ 175,000

Retrofit Cost : US$ 40,000



ANNEX C:
CHECKING DIFFERENT VULNERABILITY FACTORS OF THE BUILDING

Each of the evaluation statements on this checklist shall be marked compliant (C), non-compliant (NC), or not applicable (N/A) or not known (NK) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of FEMA 310, while non-compliant statements identify issues that require further investigation. Certain statements may not apply to the structures being evaluated. The evaluation of different statements is made and is noted by Underlined and Bold letter.

C.1. Building System

C NC N/A NK LOAD PATH: The structure shall contain one complete load path for Life Safety and Immediate Occupancy for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation.

C NC N/A NK ADJACENT BUILDINGS: An adjacent building shall not be located next to the structure being evaluated closer than 4% of the height for Life Safety and Immediate Occupancy.

C NC N/A NK WEAK STORY: The strength of the lateral-force-resisting system in any story shall not be less than 80% of the strength in an adjacent story above or below for Life-Safety and Immediate Occupancy.

C NC N/A NK SOFT STORY: The stiffness of the lateral-force-resisting system in any story shall not be less than 70% of the stiffness in an adjacent story above or below or less than 80% of the average stiffness of the three stories above or below for Life-Safety and Immediate Occupancy.

C NC N/A NK GEOMETRY: There shall be no changes in horizontal dimension of the lateral-force-resisting system of more than 30% in a story relative to adjacent stories for Life Safety and Immediate Occupancy, excluding one-story penthouses.

C NC N/A NK VERTICAL DISCONTINUITIES: All vertical elements in the lateral-force-resisting system shall be continuous to the foundation.

C NC N/A NK MASS: There shall be no change in effective mass more than 50% from one story to the next for Life Safety and Immediate Occupancy.

C NC N/A NK TORSION: The distance between the story center of mass and the story center of rigidity shall be less than 20% of the building width in either plan dimension for Life Safety and Immediate Occupancy.
Refer Annex 2 C: Check for torsion

C NC N/A NK DETERIORATION OF CONCRETE: There shall be no visible deterioration of concrete or reinforcing steel in any of the vertical- or lateral-force-resisting elements.
No such deterioration observed

C.2. Lateral Force Resisting System

C NC N/A NK REDUNDANCY: The number of lines of moment frames in each principal direction shall be greater than or equal to 2 for Life Safety and Immediate Occupancy. The number of bays of moment frames in each line shall be greater than or equal to 2 for Life Safety and 3 for Immediate Occupancy.

Meets the criteria

C NC N/A NK INTERFERING WALLS: All infill walls placed in moment frames shall be isolated from structural elements.

Infilled walls are attached to frames but not tied together

C NC N/A NK SHEAR STRESS CHECK: The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 3.5.3.2, shall be less than 100 psi or $2\sqrt{f_c}$ for Life Safety and Immediate Occupancy.

Refer Annex 2A.3: Check for shear stress

C NC N/A NK AXIAL STRESS CHECK: The axial stress due to gravity loads in columns subjected to overturning forces shall be less than 0.10f' c for Life Safety and Immediate Occupancy. Alternatively, the axial stresses due to overturning forces alone, calculated using the Quick Check Procedure of Section 3.5.3.6, shall be less than 0.30f' c for Life Safety and Immediate Occupancy.

Refer Annex 2B: Check for axial stress

C NC N/A NK FLAT SLAB FRAMES: The lateral-force-resisting system shall not be a frame consisting of columns and a flat slab/plate without beams.

Lateral force resisting system consists of columns and beams

C NC N/A NK SHORT CAPTIVE COLUMNS: There shall be no columns at a level with height/depth ratios less than 50% of the nominal height/depth ratio of the typical columns at that level for Life Safety and 75% for Immediate Occupancy.

Columns at the mid-landing of the staircases do not satisfy this criteria

CNC N/A NK NO SHEAR FAILURE: The shear capacity of frame members shall be able to develop the moment capacity at the top and bottom of the columns

Refer Annex 2A 4.2

C NC N/A NK STRONG COLUMN / WEAK BEAM: The sum of the moment capacity of the columns shall be 20% greater than that of the beams at frame joints.

Refer Annex 2D: Check for strong column weak beam

C NC N/A NK BEAM BARS: At least two longitudinal top and two longitudinal bottom bars shall extend continuously throughout the length of each frame beam. At least 25% of the longitudinal bars provided at the joints for either positive or negative moment shall be continuous throughout the length of the members for Life Safety and Immediate Occupancy.

Reference from structural drawing

C NC N/A NK COLUMN-BAR SPLICES: All columns bar lap splice lengths shall be greater than 35 d b for Life Safety and 50 d b for Immediate Occupancy and shall be enclosed by ties spaced at or less than 8 d b for Life Safety and Immediate Occupancy.

C NC N/A NK BEAM-BAR SPLICES: The lap splices for longitudinal beam reinforcing shall not be located within $l_b/4$ of the joints and shall not be located within the vicinity of potential plastic hinge locations.

C NC N/A NK COLUMN-TIE SPACING: Frame columns shall have ties spaced at or less than d/4 for Life Safety and Immediate Occupancy throughout their length and at or less than $8d_b$ for Life Safety and Immediate Occupancy at all potential plastic hinge locations.

C NC N/A NK STIRRUP SPACING: All beams shall have stirrups spaced at or less than d/2 for Life Safety and Immediate Occupancy throughout their length. At potential plastic hinge locations stirrups shall be spaced at or less than the minimum of $8d_b$ or d/4 but no less than 100mm for Life Safety and Immediate Occupancy.

As per structural drawing

C NC N/A NK JOINT REINFORCING: Beam-column joints shall have ties spaced at or less than $8d_b$ for Life Safety and Immediate Occupancy.

C NC N/A NK JOINT ECCENTRICITY: There shall be no eccentricities larger than 20% of the smallest column plan dimension between girder and column centerlines. This statement shall apply to the Immediate Occupancy Performance Level only.

C NC N/A NK STIRRUP AND TIE HOOKS: The beam stirrups and column ties shall be anchored into the member cores with hooks of 135° or more. This statement shall apply to the Immediate Occupancy Performance Level only.

As per structural drawing

C.3. Diaphragms

C NC N/A NK DIAPHRAGM CONTINUITY: The diaphragms shall not be composed of split-level floors. In wood buildings, the diaphragms shall not have expansion joints.

C NC N/A NK PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities. This statement shall apply to the Immediate Occupancy Performance Level only.

C NC N/A NK DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcing bars around all diaphragms openings larger than 50% of the building width in either major plan dimension. This statement shall apply to the Immediate Occupancy Performance Level only.

C.4. Connections

C NC N/A NK CONCRETE COLUMNS: All concrete columns shall be doweled into the foundation for Life Safety and the dowels shall be able to develop the tensile capacity of the column for Immediate Occupancy.

ANNEX D: MODIFIED MERCALLY INTENSITY SCALE (MMI Scale)

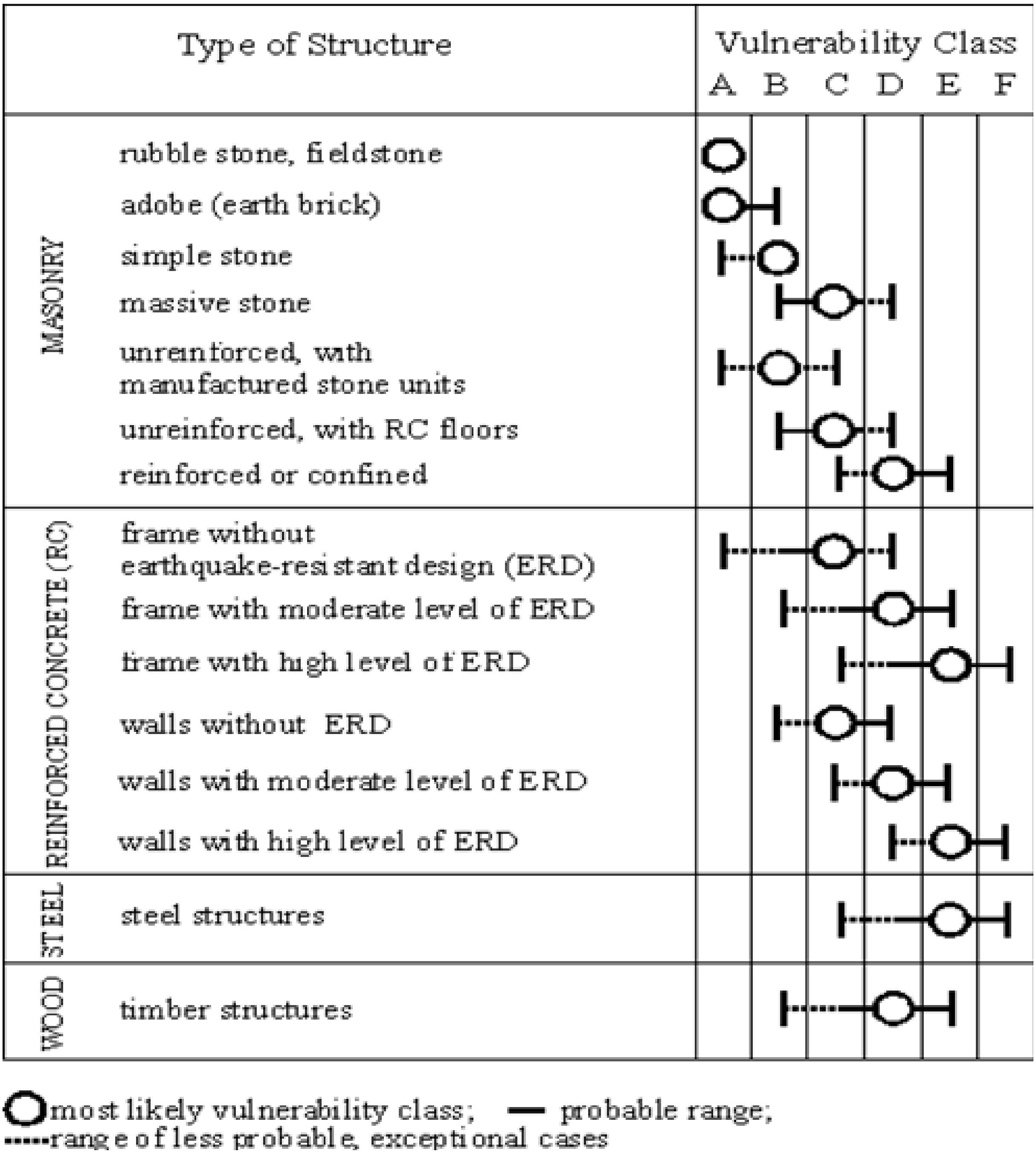
INTENSITY	DESCRIPTION OF EFFECT
I	**Very Weak Intensity** – Can only be noticed or felt by people who are in the right situation and circumstance – Furniture or things which are not correctly positioned may move or be slightly displaced – Slight shaking or vibrations will form on water or liquid surfaces in containers
II	**Slightly Weak Intensity** - Can be noticed or felt by people who are resting inside homes - Things that are hung on walls would slightly sway, shake or vibrate - The shaking or vibrations on water or liquid surfaces in containers would be highly noticeable
III	**Weak Intensity** - Can be noticed and felt by more people inside homes or buildings especially those situated at high levels. Some may even feel dizzy. The quake at this stage can be described as though a small truck has passed nearby - Things that are hung on walls would sway, shake or vibrate a little more strongly - The shaking or vibrations on water or liquid surfaces in containers would be more vigorous and stronger
IV	**Slightly Strong Intensity** - Can be noticed and felt by most people inside homes and even those outside. Those who are lightly asleep may be awakened. The quake at this stage can be described as though a heavy truck has passed nearby - Things that are hung on walls would sway, shake or vibrate strongly. Plates and glasses would also vibrate and shake, as well as doors and windows. Floors and walls of wooden houses or structures would slightly squeak. Stationary vehicles would slightly shake - The shaking or vibrations on water or liquid surfaces in containers would be very strong. It is possible to hear a slight reverberating sound from the environment
V	**Strong Intensity** - Can be felt and noticed by almost all people whether they are inside or outside structures. Many will be awakened from sleep and be surprised. Some may even rush out of their homes or buildings in fear. The vibrations and shaking that can be felt inside or outside structures will be very strong - Things that are hung on walls would sway, shake or vibrate much more strongly and intensely. Plates and glasses would also vibrate and shake much more strongly and some may even break. Small or lightly weighted objects and furniture would rock and fall off. Stationary vehicles would shake more vigorously. - The shaking or vibrations on water or liquid surfaces in containers would be very strong which will cause the liquid surfaces in containers would be very strong which will cause the liquid to spill over. Plant or tree stem, branches and leaves would shake or vibrate slightly.
VI	**Very Strong Intensity** - Many will be afraid of the very strong shaking and vibrations that they will feel causing them to loose their sense of balance and most people to run out of homes or building structures. Those who are in moving vehicles will feel as though they are having flat tyres. - Heavy objects or furniture would be displaced from original positions. Small hanging bells would shake and ring. Outer surfaces of concrete walls may crack. Old or fragile houses, buildings or structures would be slightly damaged. - Weak to strong landslides may occur. The shaking and vibration of plant or tree stem, branches and leaves would be strong and highly noticeable.

INTENSITY	DESCRIPTION OF EFFECT
VII	**Damaging Intensity** - Almost all people will be afraid of the very strong shaking and vibrations that they will feel. Those who are situated at high levels of building swill find it very hard to keep standing. - Heavy objects or furniture would fall and topple over. Large hanging bells will sound vigorously. Old or fragile houses, buildings or structures would most definitely be destroyed, while strong or new structures would be damaged. Dikes, dams, fishponds, concrete roads and walls may crack and be damaged. - Liquefaction (formation of quicksand), lateral spreading (spreading of soil surface creating deep cracks on land) and landslides will occur. Trees and plants will vigorously shake and vibrate.
VIII	**Highly Damaging Intensity** - Will cause confusion and chaos among the people. It makes standing upright difficult even outside homes/structures. - Many big buildings will be extremely damaged. Landslides or lateral spreading will cause may bridges to fall and dikes to be highly damaged. It will also cause train rail tracks to bend or be displaced. Thombs will be damaged or be out of place. Posts, towers and monuments may bend or completely be destroyed. Water and canal/drainage pipes may be damaged, bend or break. - Liquefaction and lateral spreading causes structures to sink, bend or be completely destroyed, especially those situated on hills and mountains. For places near or situated at the earthquake epicenter, large stone boulders may be thrown out of opposition. Cracking, splitting, fault rupture of land may be seen. Tsunami-like waves will be formed from water surfaces whether from rivers, ponds or dams/dikes. Trees and plant life will very vigorously more and sway in all directions.
IX	**Destructive Intensity** - People would be forcibly thrown/fall down. Chaos, fear and confusion will be extreme - Most building structures would be destroyed and intensely damaged. Bridges and high structures would fall and be destroyed. Posts, towers and monuments may bend or break. - Landslides, liquefaction, lateral spreading with sand boil (rise of underground mixture of sand and mud) will occur in many places, causing the land deformity. Plant and trees would be damaged or uprooted due to the vigorous shaking and swaying. Large stone boulders may be thrown out of position and be forcibly darted to all directions. Very-very strong tsunami-like waves will be formed from water surfaces whether from rivers, ponds or dams/dikes.
X	**Extremely Destructive Intensity** - Overall extreme destruction and damage of all man-made structures - Widespread landslides, liquefaction, intense tsunami like waves formed will be destructive. There will be tremendous chance in the flow of water on rivers, springs, and other waterforms. All plant life will be destroyed and uprooted.
XI	**Devastative Intensity** • Severe damage even to well built buildings, bridges, water dams and railway lines; highways become useless; underground pipes destroyed.
XII	**Extremely Destructive Intensity (Landscape changes)** • Practically all structures above and below ground are greatly damaged or destroyed.

ANNEX E: EUROPEAN MACRO SEISMIC SCALE (EMS 98)

Classifications used in the European Macro seismic Scale (EMS)

Differentiation of structures (buildings) into vulnerability classes
(Vulnerability Table)



ANNEX F: EQUIPMENT USED IN RETROFITTING

Some of the major equipment used in retrofitting are:

1. Schmidt hammer

A Schmidt hammer, also known as a Swiss hammer or a rebound hammer, is a device to measure the elastic properties or strength of concrete or rock, mainly surface hardness and penetration resistance.

It was invented by Ernst Schmidt, a Swiss engineer.

The hammer measures the rebound of a spring-loaded mass impacting against the surface of the sample. The test hammer will hit the concrete at a defined energy. Its rebound is dependent on the hardness of the concrete and is measured by the test equipment. By reference to the conversion chart, the rebound value can be used to determine the compressive strength. When conducting the test the hammer should be held at right angles to the surface which in turn should be flat and smooth. The rebound reading will be affected by the orientation of the hammer, when used in a vertical position (on the underside of a suspended slab for example) gravity will increase the rebound distance of the mass and vice versa for a test conducted on a floor slab. The Schmidt hammer is an arbitrary scale ranging from 10 to 100. Schmidt hammers are available from their original manufacturers in several different energy ranges. These include: (i) Type L-0.735 Nm impact energy, (ii) Type N-2.207 Nm impact energy; and (iii) Type M-29.43 Nm impact energy.

The test is also sensitive to other factors:

- Local variation in the sample. To minimize this it is recommended to take a selection of readings and take an average value.
- Water content of the sample, a saturated material will give different results from a dry one.

Prior to testing, the Schmidt hammer should be calibrated using a calibration test anvil supplied by the manufacturer for that purpose. 12 readings should be taken, dropping the highest and lowest, and then take the average of the ten remaining. Using this method of testing is classed as indirect as it does not give a direct measurement of the strength of the material. It simply gives an indication based on surface properties; it is only suitable for making comparisons between samples.

2. Bar Scanner

To determine the position, depth and diameter of rebar can be problematic in everyday construction work. The bar scanner was developed is a portable, quick and simple-to-operate system that solves all these problems and many more:

- Finding a secure drilling point for drilling or coring work
- Carrying out structural analyses quickly and exactly in a non-destructive manner
- Determining coverage over the entire surface of a structure

3. Grouting Machine

Pressure grouting involves injecting a grout material into generally isolated pore or void space of which neither the configuration nor volume are known, and is often referred to simply as grouting. The grout may be a cementitious, resinous, or solution chemical mixture. The greatest use of pressure grouting is to improve geomaterials (soil and rock). The purpose of grouting can be either to strengthen or reduce water flow through a formation. It is also used to correct faults in concrete and masonry structures. Since first usage in the 19th century, grouting has been performed on the foundation of virtually every one of the world's large dams, in order to reduce the amount of leakage through the rock, and sometimes to strengthen the foundation to support the weight of the overlying structure, be it of concrete, earth, or rock fill. Although very specialized, pressure grouting is an essential construction procedure that is practiced by specialist contractors and engineers around the world.

4. Drilling Machine

A drill is a tool fitted with a cutting tool attachment or driving tool attachment, usually a drill bit or driver bit, used for boring holes in various materials or fastening various materials together with the use of fasteners. The attachment is gripped by a chuck at one end of the drill and rotated while pressed against the target material. The tip, and sometimes edges, of the cutting tool does the work of cutting into the target material. This may be slicing off thin shavings (twist drills or auger bits), grinding off small particles (oil drilling), crushing and removing pieces of the workpiece (SDS masonry drill), counter sinking, counter boring, or other operations.

Drills are commonly used in woodworking, metalworking, construction and do-it-yourself projects. Specially designed drills are also used in medicine, space missions and other applications. Drills are available with a wide variety of performance characteristics, such as power and capacity.

ANNEX G: OVERVIEW OF SOME DAMAGED RC BUILDINGS AND ITS CAUSE

Strong beam weak column

Weak Beam- column Joint

Soft Upper-Storey

Soft Ground Storey

Weak Beam - Column Joint

Unequal settlement

Due to confinement

Weak storey

Figure A-44 Photos of different Damage patterns (Source: Dr. Hari Darshan Shrestha)

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SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL

RCC STRUCTURES

Empowered lives. Resilient nations.