SEISMIC RETROFITTING GUIDELINES

OF

BUILDINGS IN NEPAL

2013

RCC

UNDP

CORD

MRB & Associates

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S E I S M I C RETROFITTING GUIDELINES of Buildings in Nepal

RCC STRUCTURES

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1. INTRODUCTION

This report has been prepared by MRB &Associates and CoRD Nepal as part of content for Guideline of Retrofitting of RCC structures.

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 evidence of continued uplift. As a result, Nepal is very active seismically. 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 built before the code implemented can be rebuild or reconstruct to gain 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.

2. CONCEPT OF REPAIR, RESTORATION AND RETROFITTING

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.

2.1 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. After anchorages are installed, 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.

Figure 2-1 Anchors driven inside concrete after placing epoxy resin

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.

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.

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 ren 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.

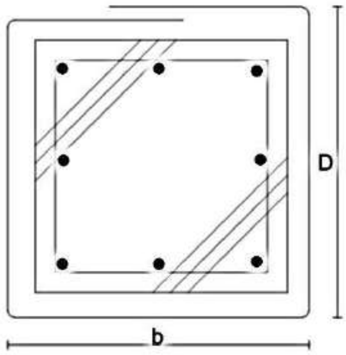


Figure 2-5 **Cross- section of Column**

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.

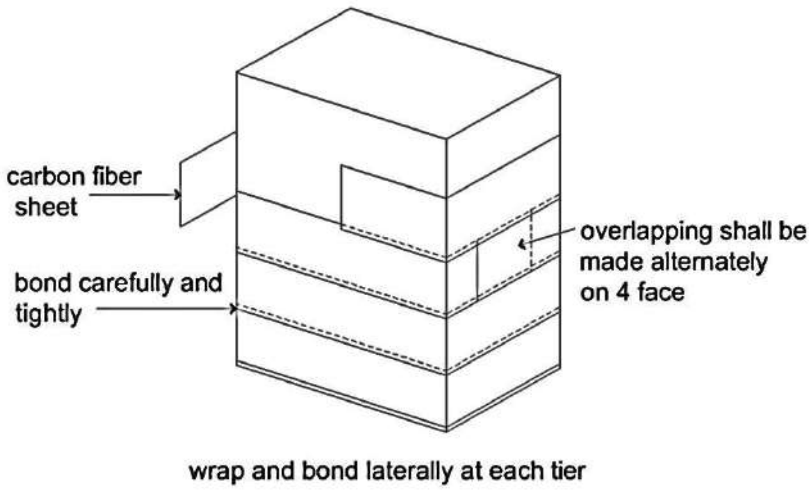


Figure 2-6 Strengthening with carbon fiber sheet wrapping

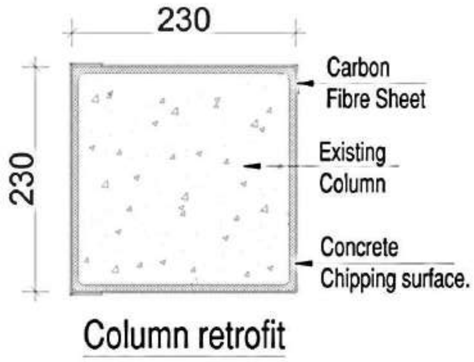


Figure 2-7 Column retrofit

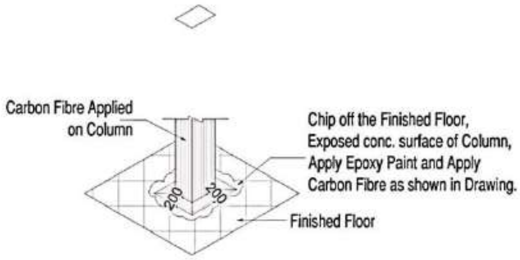


Figure 2-8 3D view of carbon fibre applied in column at floor level

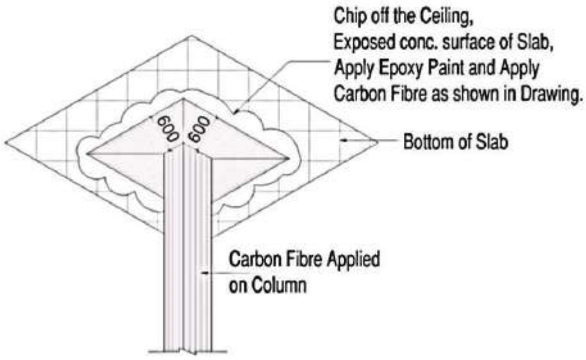
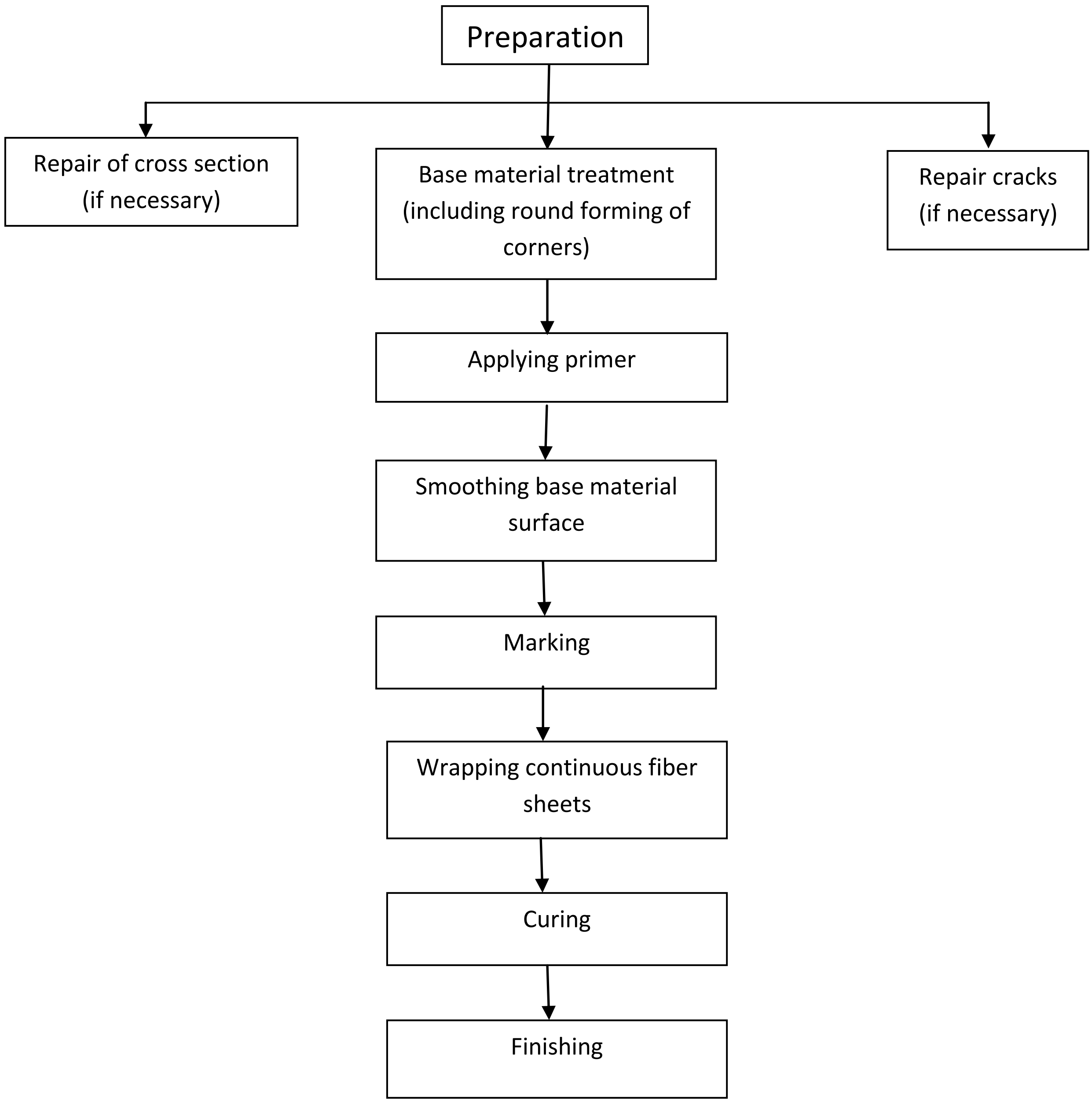


Figure 2-9 3D view of carbon fibre applied in column at ceiling level

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 –threating 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.

3.2.3 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:

- Age of building
- Structural System – Load bearing Or Frame Structure
- Foundation Exploration
- Load path
- Geometry
- Walls Detail – Size and mortar

- Beam and Column Size
- Water proofing method
- Renovation of Building
- 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:

The stiffness of vertical lateral load resisting system in any storey shall not be less than 60% of the stiffness in an adjacent storey or less than 70% of the average stiffness of the three storey 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 Torsion:

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).

a. Calculation of earthquake loads using Seismic coefficient method:

The design horizontal seismic coefficient, $\mathbf{A_h = Z*I*S_a/2Rg}$

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 :

$$\mathbf{V_B = A_h*W}$$

Where, Ah = 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:

$$\mathbf{T_a = 0.09h / d^{0.5}}$$

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

b. Distribution of base shear and calculation of shear stress in RC Columns

The design base shear (VB) computed in 1.5 shall be distributed along the height of the building as per the following expression:

$$\mathbf{Q_i = V_B \cdot (W_i h_i^2 / \sum W_i h_i^2)}$$

Where,

Q_i = Design lateral force at floor *i*,

W_i= Seismic weight of floor *i*,

h_i= Height of floor *i* measured from base

c. Shear Stress in RC Frame Columns

Average Shearing stress in columns is given as

$\tau_{col} = (n_c/(n_c-n_f)) * (V_j / A_c)$< min of 0.4 Mpa and 0.1 sq.rt.(f_{ck})

$0.1\sqrt{fck}$ = 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

d. Axial Stress Check:

Axial stresses due to overturning forces as per FEMA 310

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 Fo is given by

Fo = 2/3 (Vb/nf) x (H / L)

Vb = Base shear x Load Factor

Ac = column area

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 4-1 Schimdt Hammer (source: MRB & Associates)

Figure 4-2 Ferro scanner (source: MRB & Associates)

Figure 4-3 Ultra sonic range finder

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 themaximum of:

a) Calculated factored shear force as per analysis,

b) a factored shear force given by,

$V_u = 1.4\ (M1 + m1')/\ h_{st}$

M1 and m1' 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 shall be at least 1.1 times the sum of the moment of resistance of the beams at each frame joint.***

$\sum M_c \geq 1.1 \sum M_b$

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, short columns

<u>Seismic preliminary evaluation</u>

- 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?

Yes

No

<u>Detailed evaluation</u>

- Calculation of moment of resistance in hogging and sagging.
- Check of shear capacity of Beam and Column.

Acceptability criteria Satisfied?

Yes

No

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

Selection of most appropriate retrofitting option

- Detail drawing & report
- Relatively Safe Area drawing

Construction, Quality Assurance and Control

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.	
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.	
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.	
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

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

5.1.1 DAMAGE GRADE 1

Figure 5-0-1 Damage grade 1 (source: MRB & Associates)

5.1.2 DAMAGE GRADE 2

Figure 5-0-2 Damage grade 2 (source: MRB & Associates)

5.1.3 DAMAGE GRADE 3

Figure 5-0-3Damage grade 3 (source: MRB & Associates)

5.1.4 DAMAGE GRADE 4

Figure 5-0-4 Damage grade 4 (source: MRB & Associates)

5.1.5 DAMAGE GRADE 5

Figure 5-0-5 Damage grade 5 (source: MRB & Associates)

Figure 5-0-6 Damage grade 5 (source: MRB & Associates)

5.2 OVERVIEW OF SOME DAMAGED RC BUILDINGS AND ITS CAUSE

Damage due to torsion

Figure 0-7 Damage dure to torsion

Weak Storey

Soft Upper-Storey

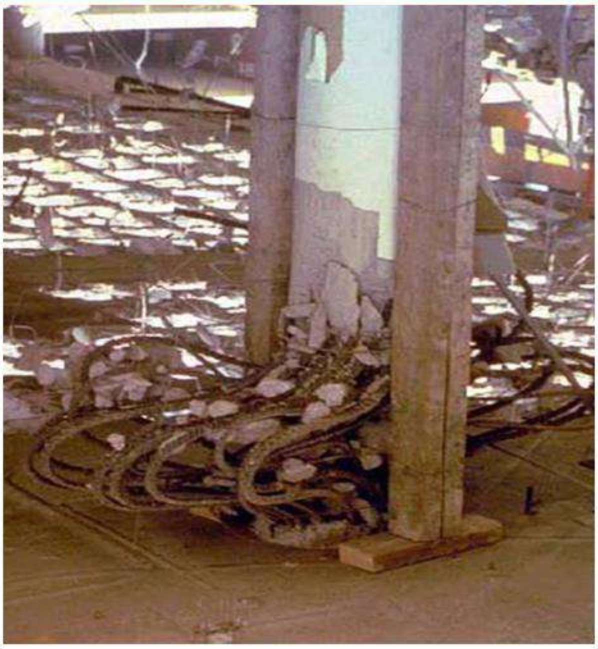
Soft Ground Storey

Short Column due to parapet wall

Short Column due to partial masonry wall

Pounding due to adjacent building

Pounding due to adjacent building



Column Tie spacing and hook

Column Tie spacing and hook

Strong Column Weak Beam

Strong Column Weak Beam

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 raise 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.

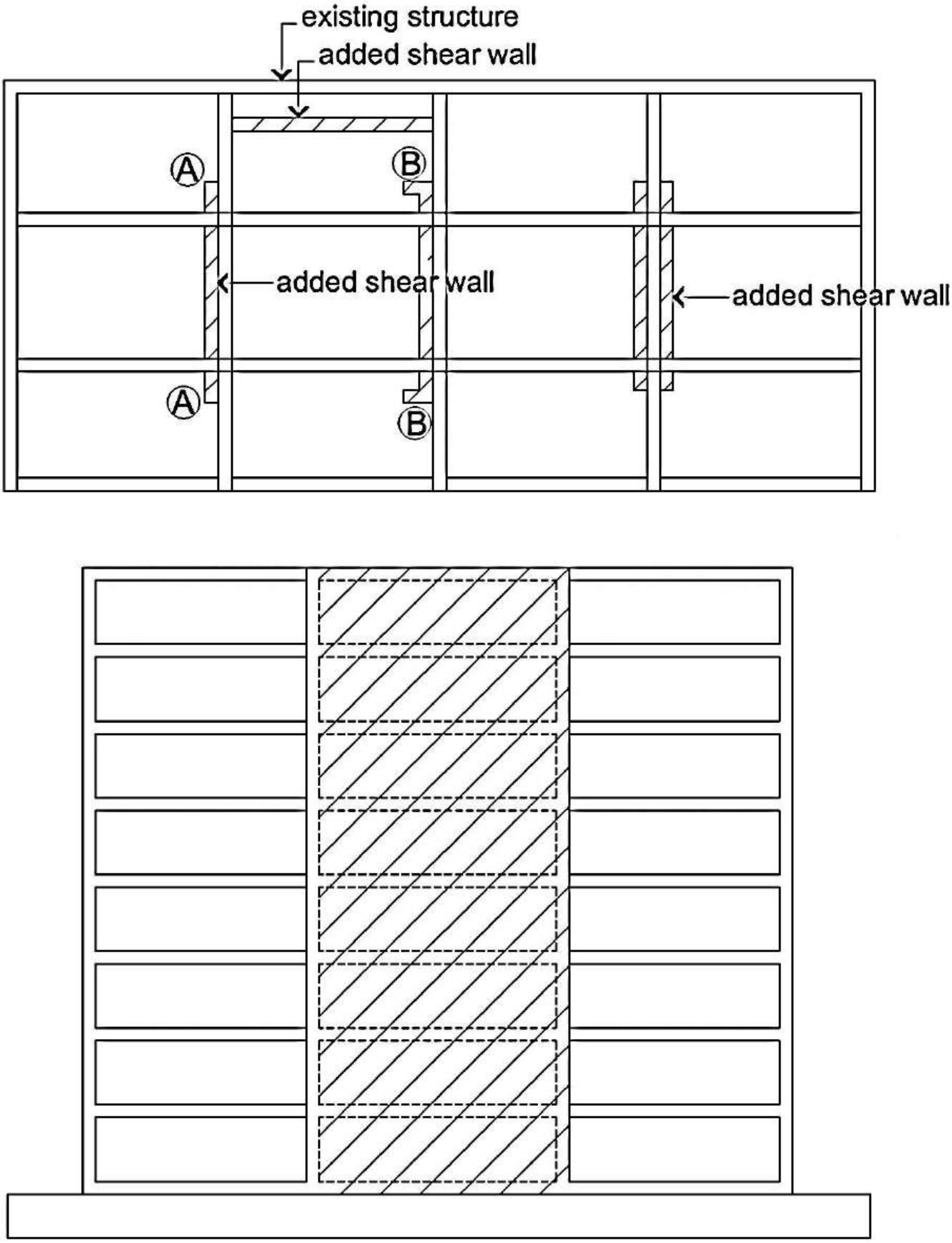


Figure 6-1 Shear wall in existing structure

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

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

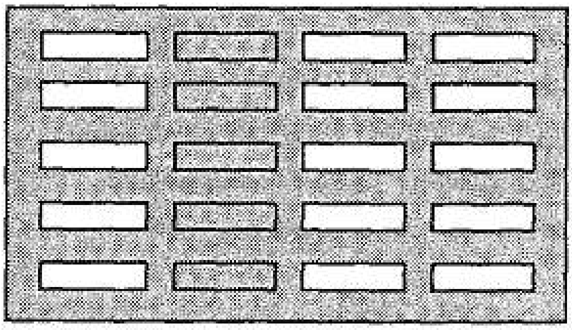


Figure 6-3 Building retrofit with infill windows

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.

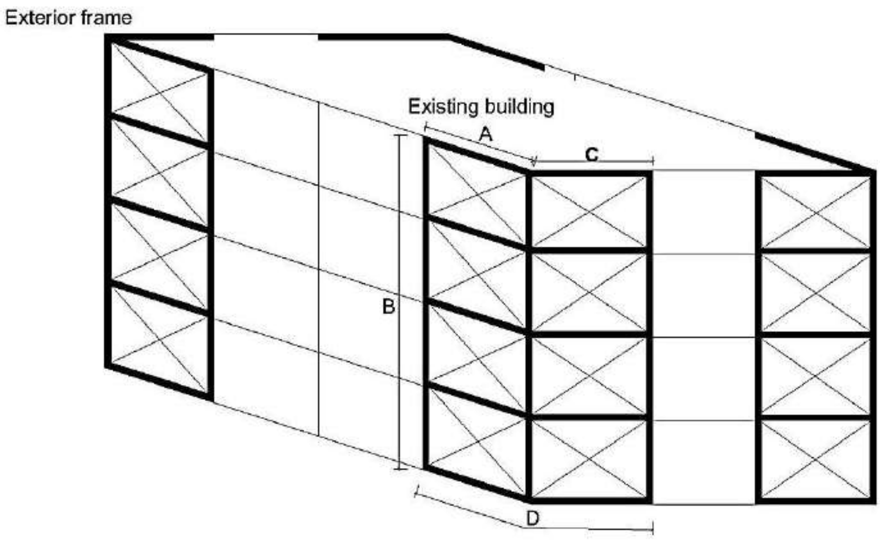


Figure 6-4 Exterior frame (steel framed brace)

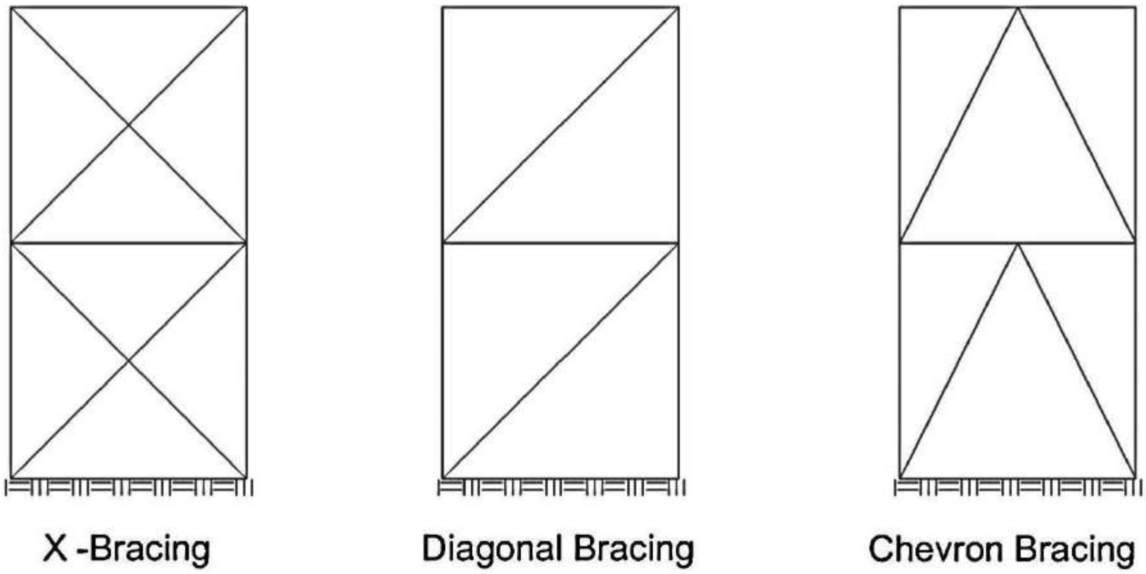


Figure 6-5 Types of bracings

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.

Roughening of old surface by chipping of existing concrete surface

Anchorage of new reinforcement with the existing structure

Welding of new reinforcement with existing one

Quality control by testing concrete grade

Figure 6-6 Strengthening of original structure

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, bucking 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.

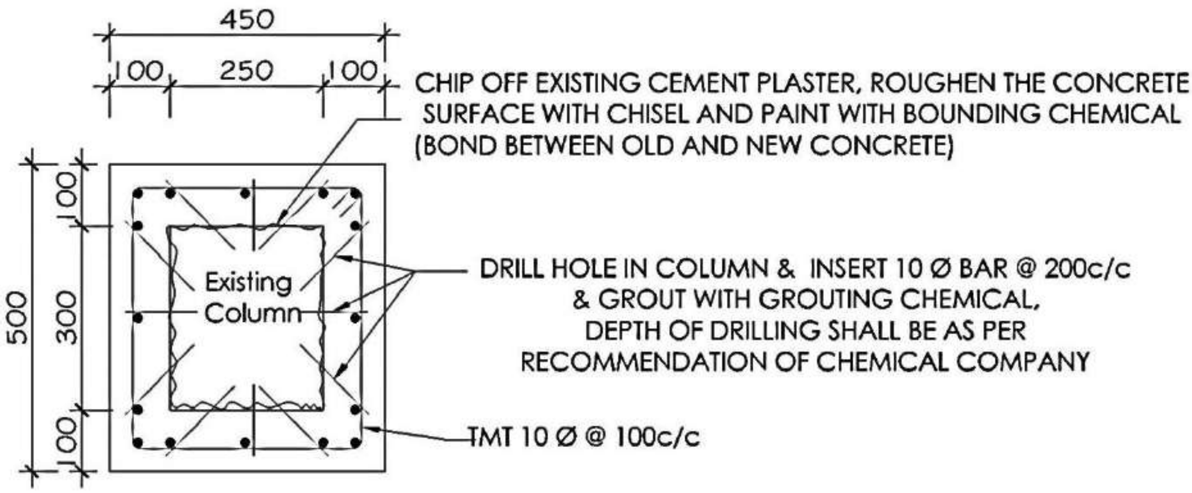


Figure 6-7 Column RC jacketing plan

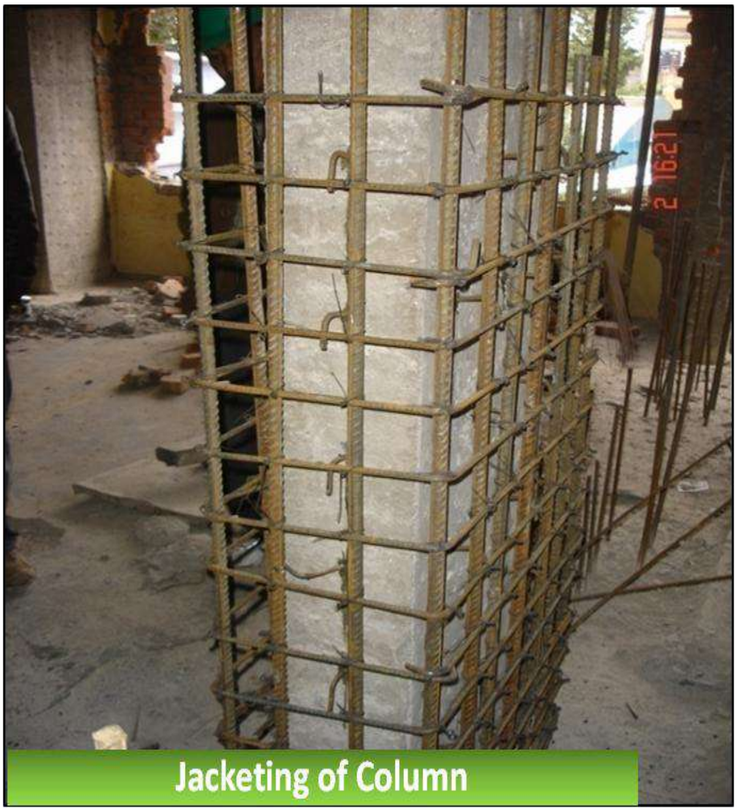


Figure 6-8 Jacketing of cloumn

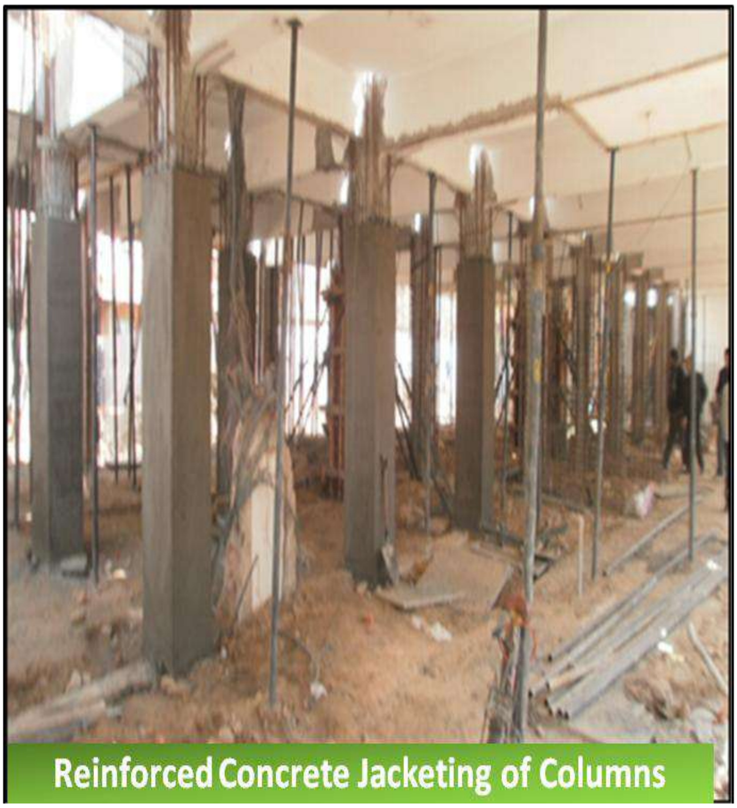
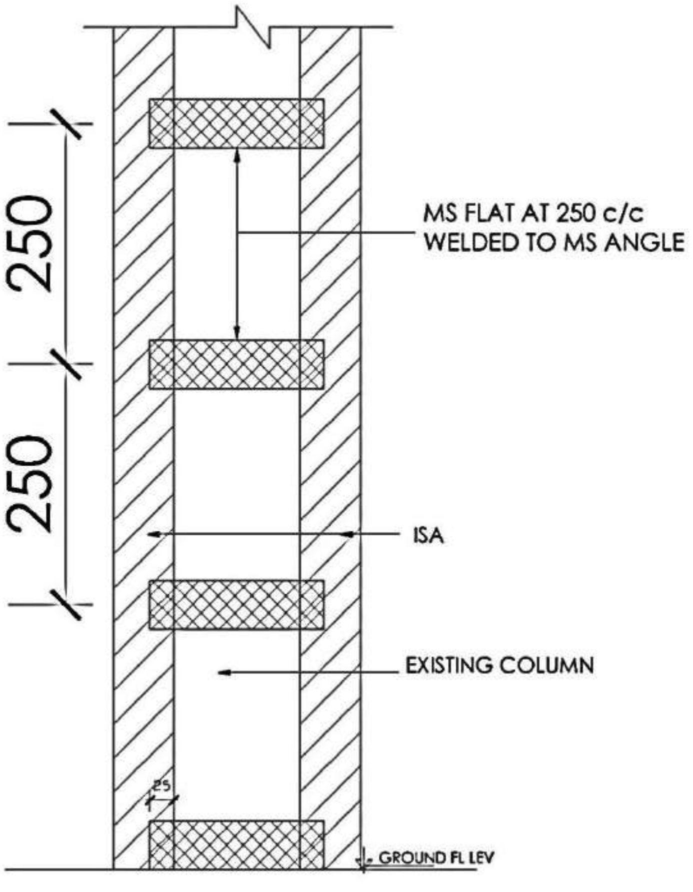


Figure 6-9 RC jackeing of columns



COLUMN STEEL JACKETING ELEVATION

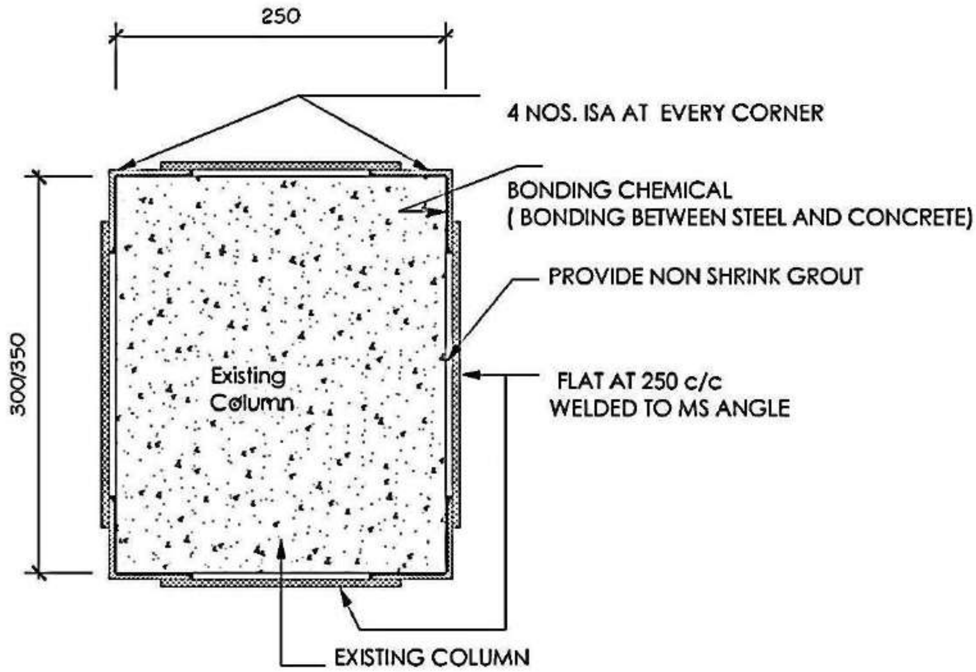


Figure 6-10 Cloumn steel jacketing plan

Figure 6-11 Steel jacketing of columns (source: MRB & Associates)

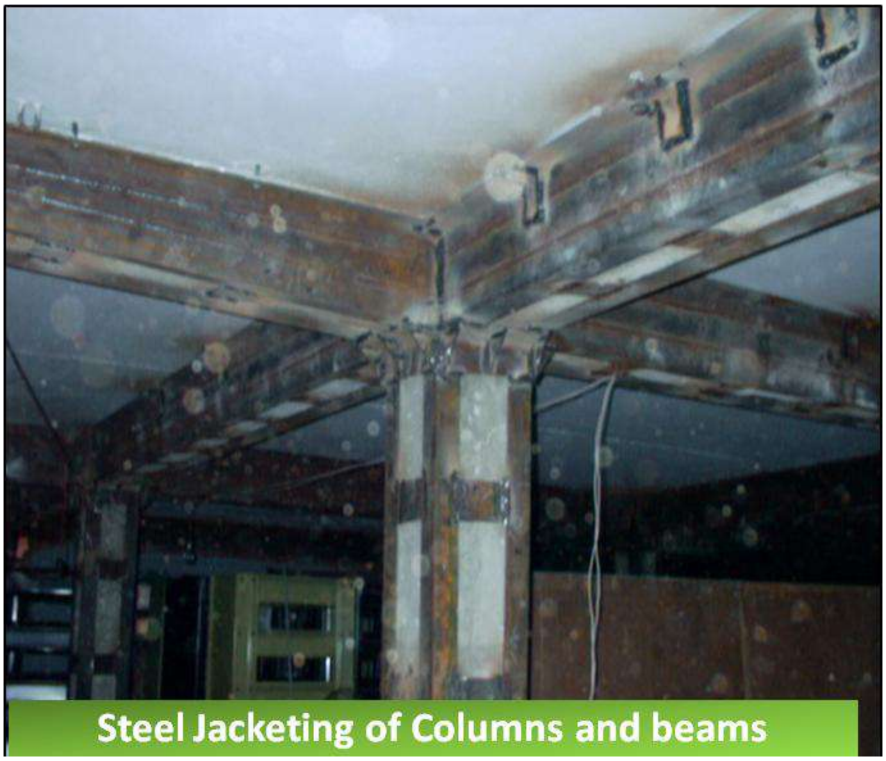


Figure 6-12 Steel jacketing of columns and beams (source: MRB & Associates)

- 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.

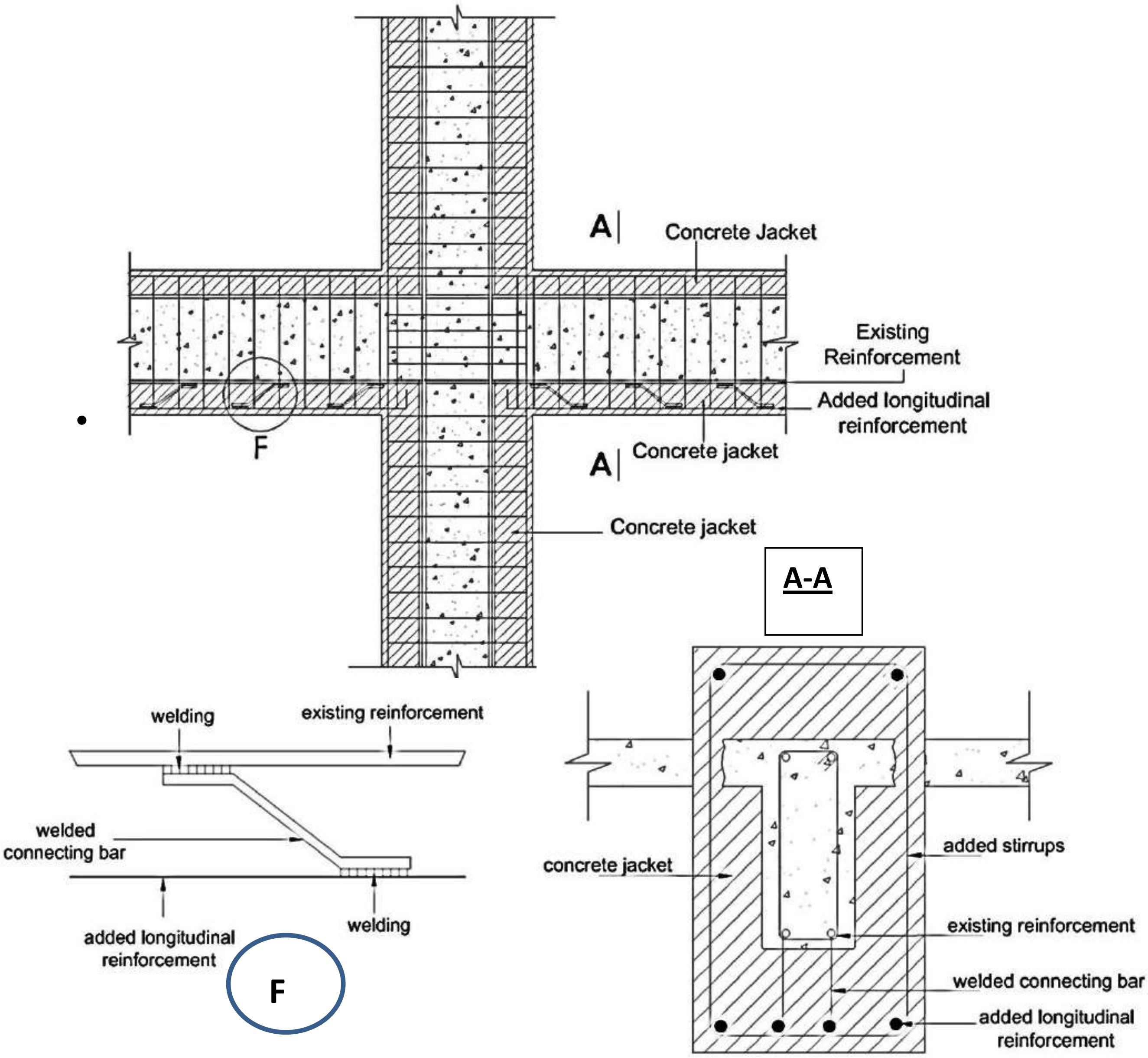


Figure 6-13 RC jacketing of beam

Figure 6-14 Reinforcement placing for beam jacketing

Figure 6-15 Top reinforcement detailing of beam jacketing

Figure 6-16 Beam Jacket (source: MRB & Associates)

Figure 6-17 Encasement fo 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.

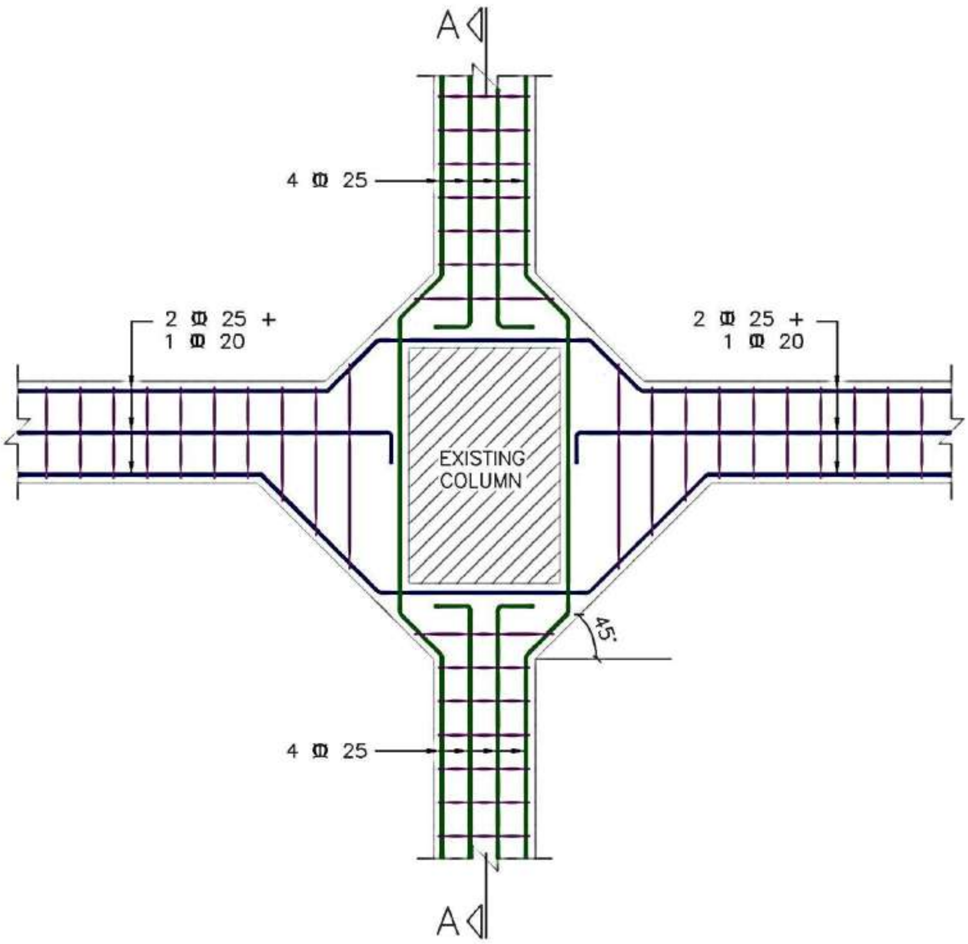


Figure 6-18 Example of beam column joint

- 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.

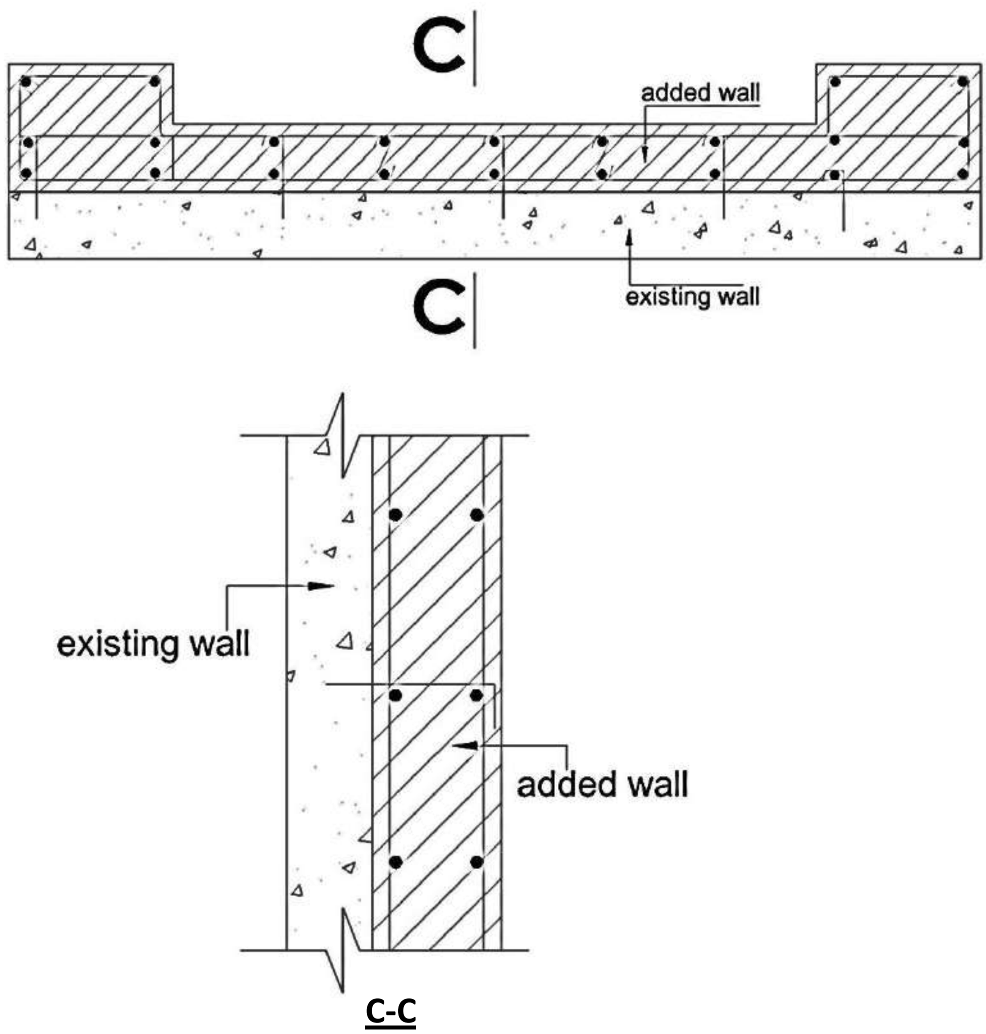


Figure 6-19 Example of shear wall retrofit

- 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.

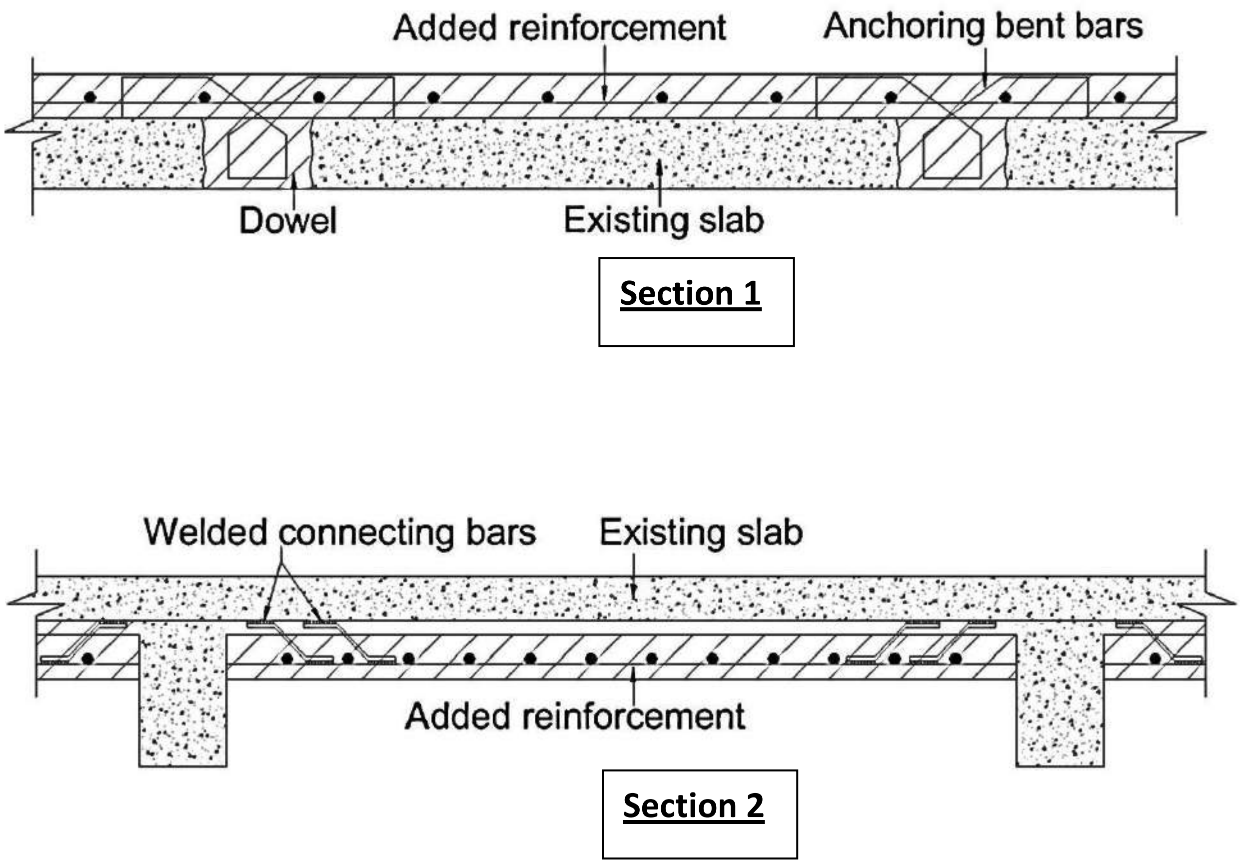


Figure 6-20 Increasing slab thickness

- 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 resit, 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 are 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.

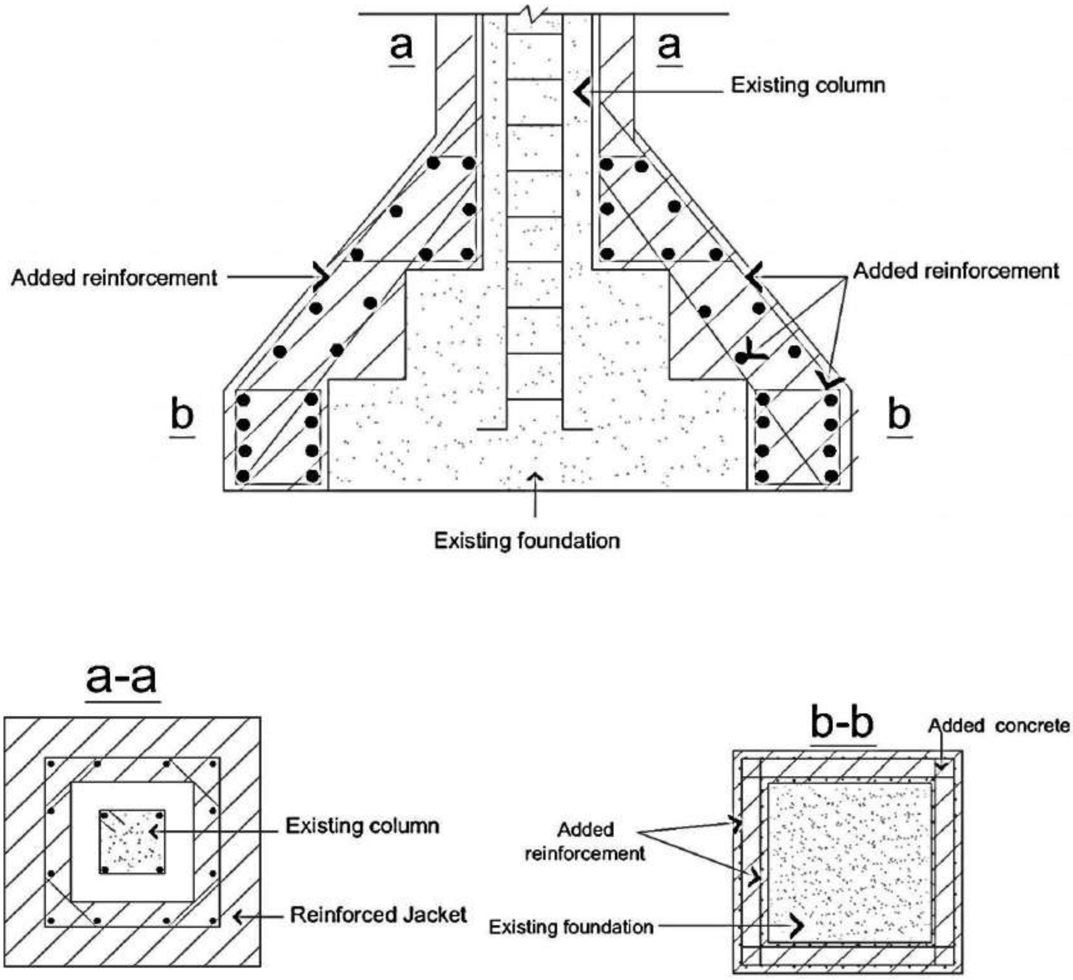


Figure 6-21 Foundation retrofit

Figure 6-22 Reinforcement layout at foundation for retrofit (source: MRB & Associates)

Figure 6-23 Reinforcement layout at foundation for retrofit (source: MRB & Associates)

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.

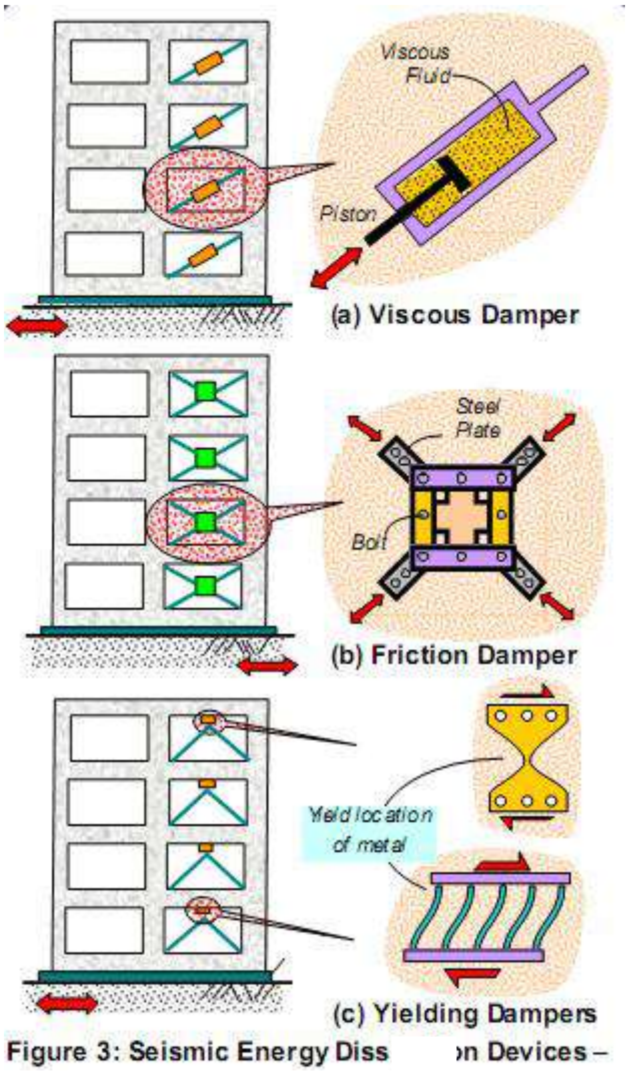


Figure 6-25 Energy dissipation system

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 reinforce 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.

   $Ac = 1.5\ Ac'$ and $As = 4/3\ As'$

   Where, Ac and As = Actual concrete and steel to be provide in the jacket

   Ac′ and As′ = 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:

   $s = fy\ dh2/\sqrt{fck}.tj$

   where

   fy= yield strength of steel

   fck= cube strength of concrete

   dh= diameter of stirrup

   tj= 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 90º 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 minimum of 12φ bars in the four corners and ties of8φ @ 100 c/c should be provided with135º 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 200mm whichever is less.

Option 1:

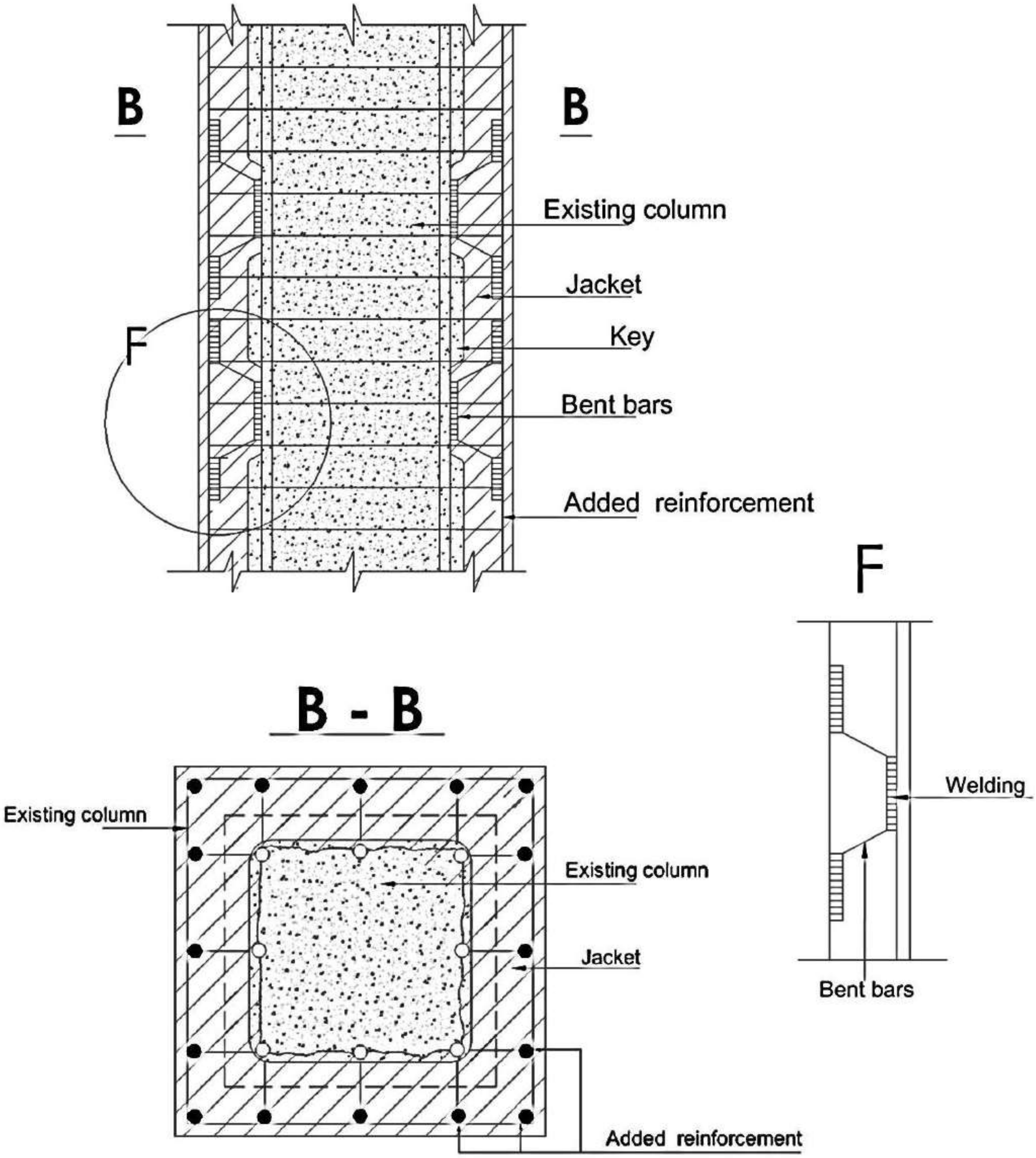


Figure 6-26 Column jacketing with reinforced concrete- option 1

Option 2:

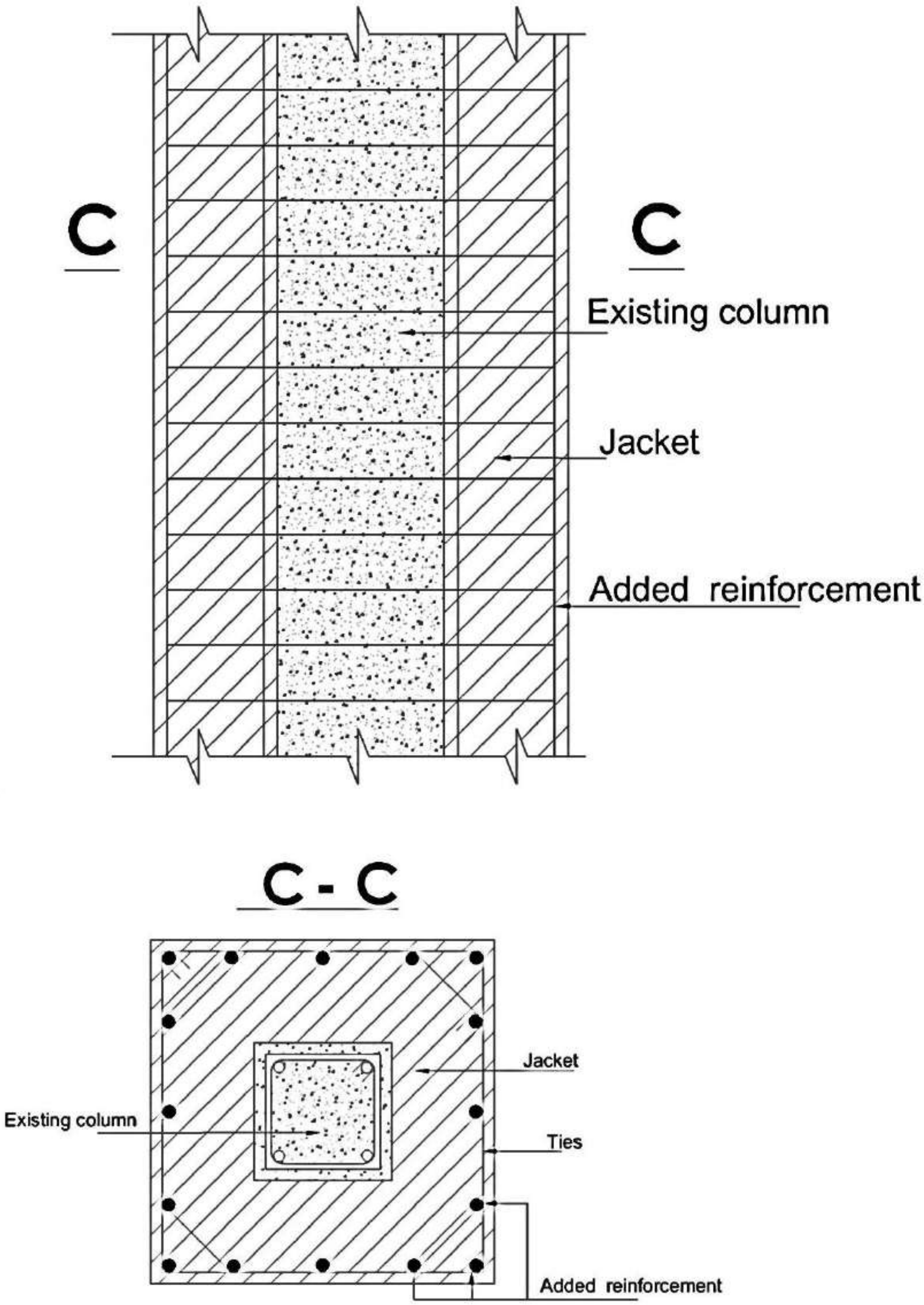


Figure 6-27 Column jacketing with reinforced concrete- option 2

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 50X50X5. 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.

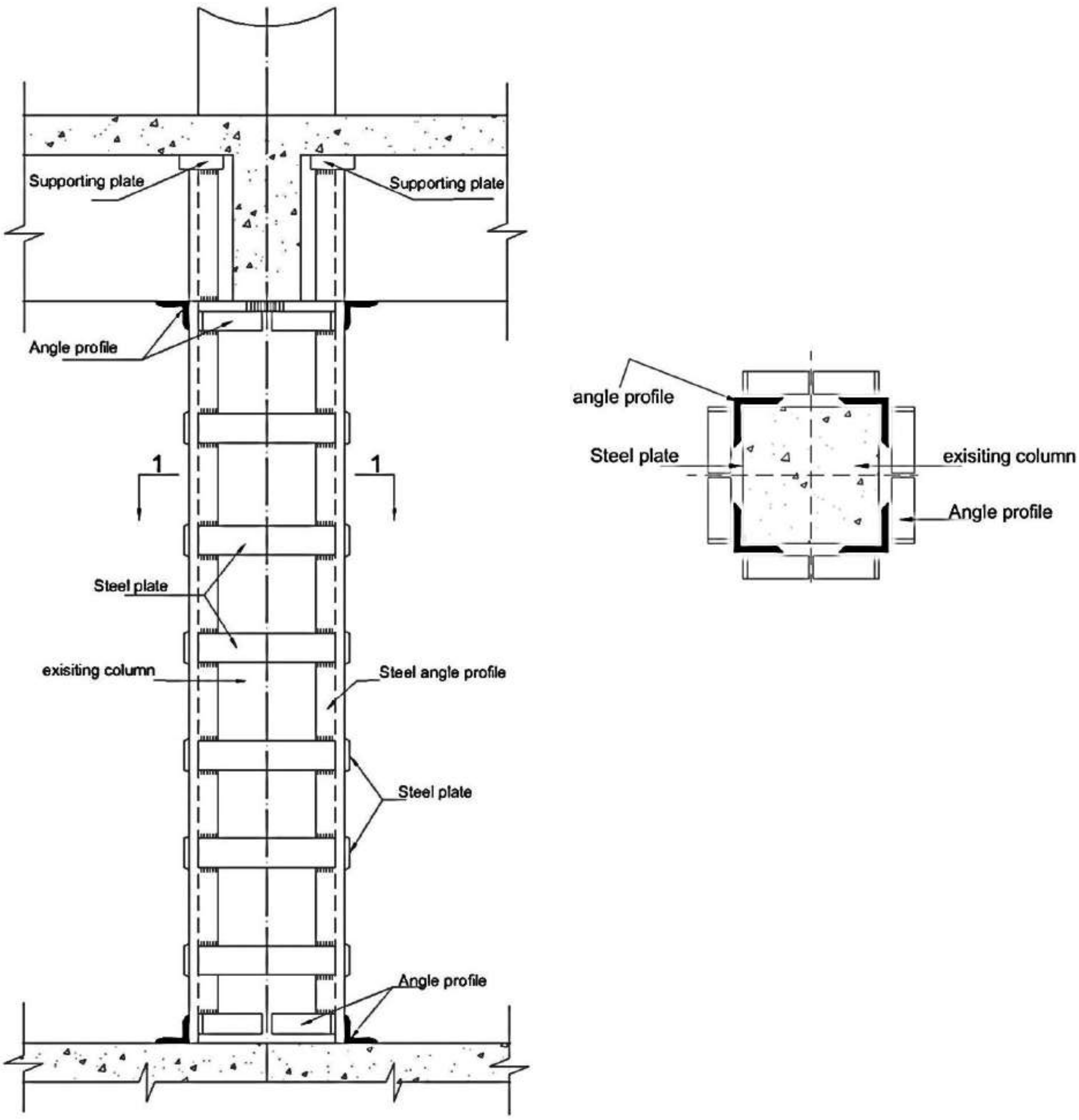


Figure 6-28 Steel jacketing of columns

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.

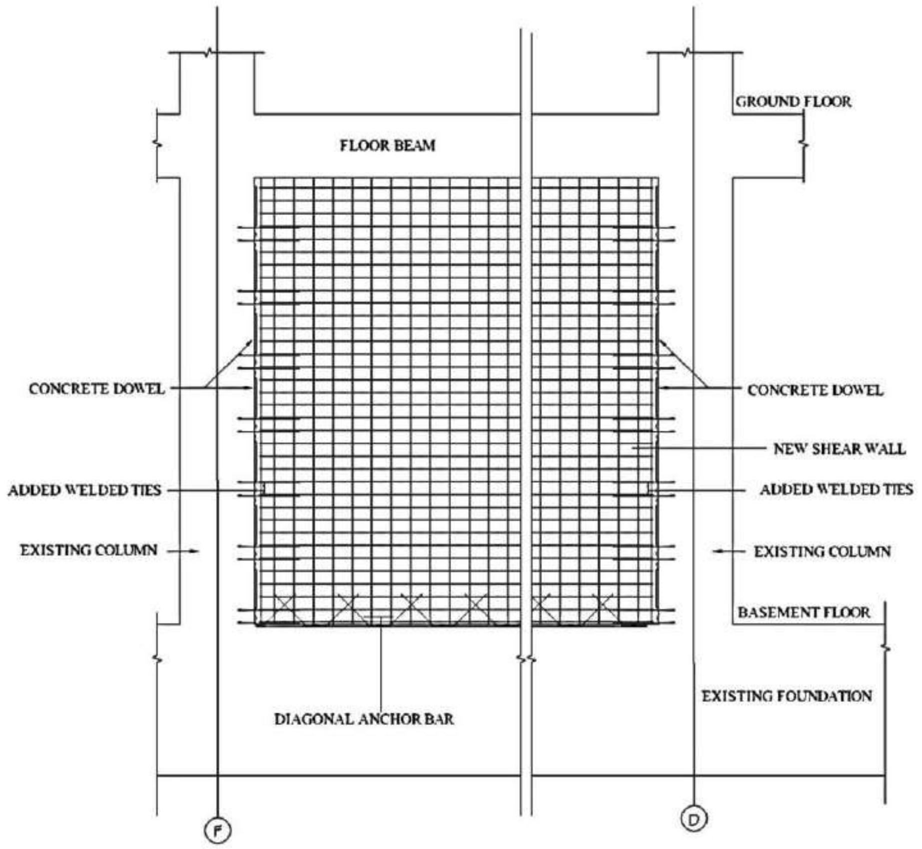


Figure 6-29 Addition of shear wall

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.

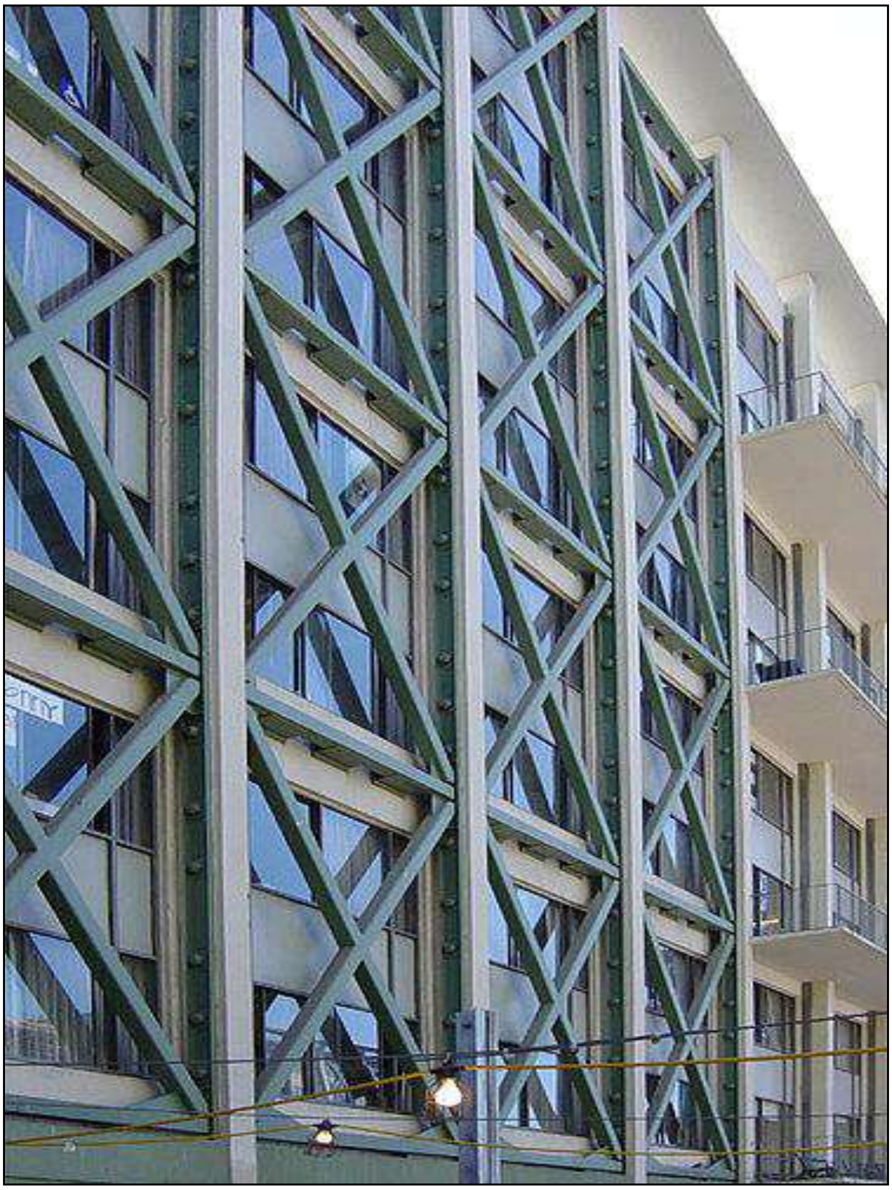
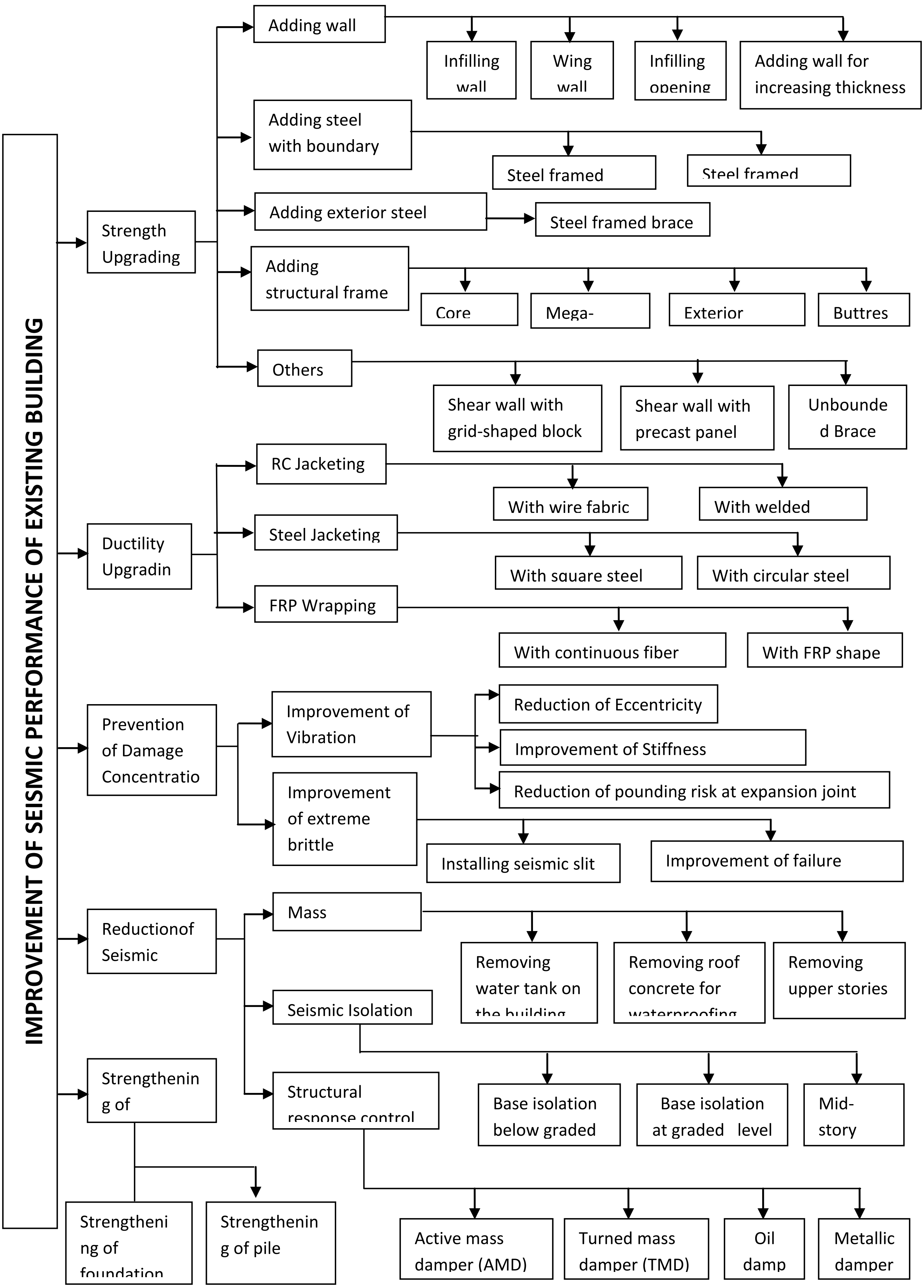


Figure 6-31 Addition of steel bracing


IMPROVEMENT OF SEISMIC PERFORMANCE OF EXISTING BUILDING
Strength Upgrading
Adding wall
Infilling wall
Wing wall
Infilling opening
Adding wall for increasing thickness
Adding steel with boundary
Steel framed
Steel framed
Adding exterior steel
Steel framed brace
Adding structural frame
Core
Mega-
Exterior
Buttres
Others
Shear wall with grid-shaped block
Shear wall with precast panel
Unbounded Brace
Ductility Upgradin
RC Jacketing
With wire fabric
With welded
Steel Jacketing
With square steel
With circular steel
FRP Wrapping
With continuous fiber
With FRP shape
Prevention of Damage Concentratio
Improvement of Vibration
Reduction of Eccentricity
Improvement of Stiffness
Reduction of pounding risk at expansion joint
Improvement of extreme brittle
Installing seismic slit
Improvement of failure
Reductionof Seismic
Mass
Removing water tank on the building
Removing roof concrete for waterproofing
Removing upper stories
Seismic Isolation
Base isolation below graded
Base isolation at graded level
Mid-story
Structural response control
Active mass damper (AMD)
Turned mass damper (TMD)
Oil damp
Metallic damper
Strengthening of
Strengthening of foundation
Strengthening of pile

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)

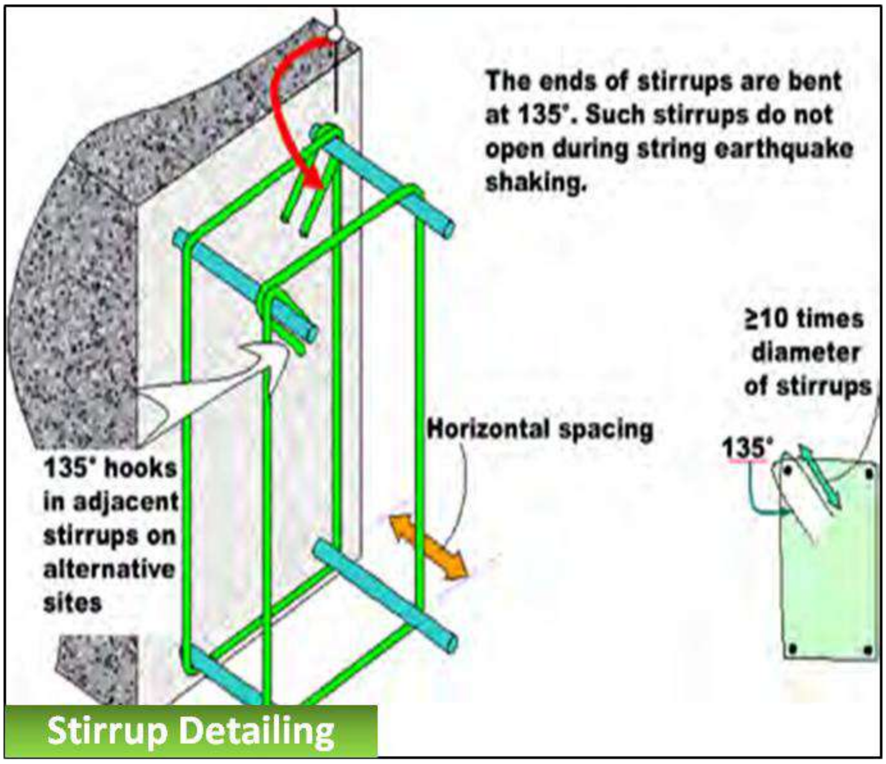


Figure 6-34 Stirrup detailing

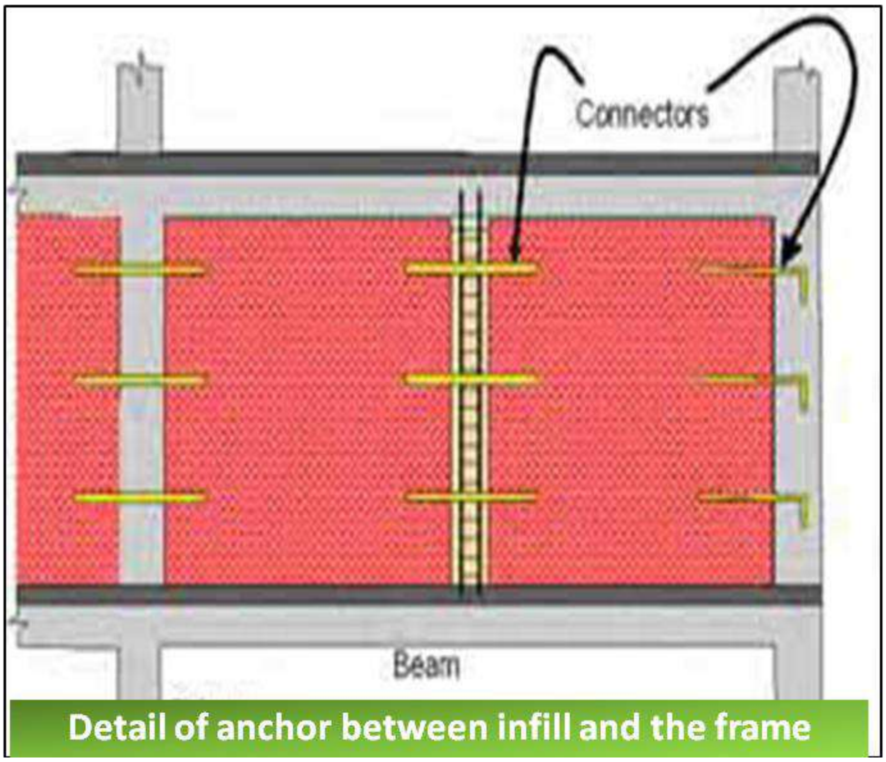


Figure 6-35 Detail of anchor between infill and the frame

7. VULNERABILITY ANALYSIS

Vulnerability analysis is very important to protect building or structures from damage. The vulnerability assessment is necessary due to many reasons. 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 vulnerability assessment and retrofit of Engineered building has been demonstrated in this guideline as Example no. 1 and that for occupancy change building has been demonstrated as Example no. 2

7.1 EXAMPLE NO. 1

ENGINEERED RC FRAME BUILDING

7.1.1 BUILDING DESCRIPTION

This building is RCC Frame structure in burnt clay bricks in cement mortar. The structure is 5-story + 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. But it was built before the new code was introduced to Nepal and one minus point of the building is that, it is L- shaped, which is not favorable for earthquake. The building has been assessed using Nepal Building Code as per the request of client.

Vulnerability analysis was done and retrofitting is recommended to fulfill new building code and 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.

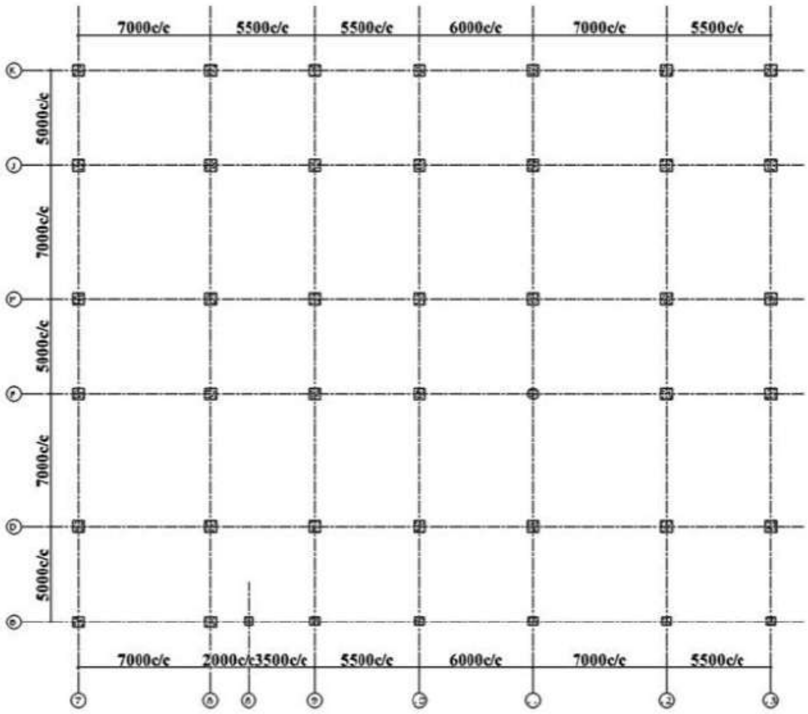
7.1.1.1 General Building Description

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

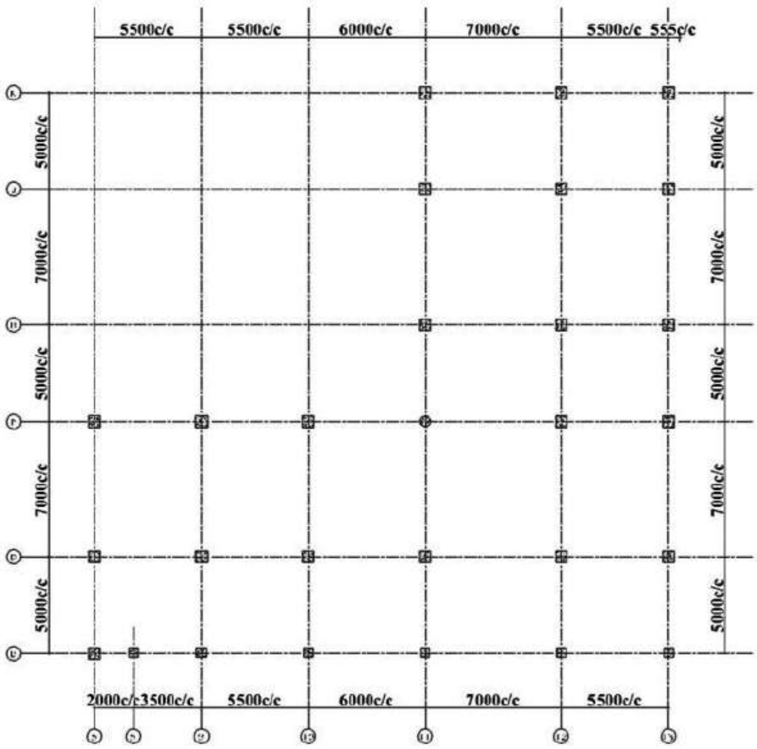
7.1.1.2 Structural System Description

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

7.1.1.3 Building Drawings



BASEMENT PLAN



TYPICAL FLOOR PLAN

Figure 7-1 Building drawings

7.1.2 ASSUMPTIONS

Unit weight of RCC= 25 kN/m3

Unit weight of brick = 19.6 kN/m3

Live load = 3.0 kN/m2

Weight of plaster and floor finish = 0.73 kN/m2 i.e. 22mm screed + 12mm plaster

Partition load = 1.2 kN/m2

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.

7.1.3 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.*

7.1.4 STRENGTH RELATED CHECKS

7.1.4.1 Calculation For Shear Stress Check

Lumped load

LEVEL	Combination DL+0.25LL	Seismic Weight
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

∑ **46459.57** **kN**

7.1.4.2 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 = Impotance 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:

$$T_a = 0.09h / d^{0.5}$$

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

$T_{ax} = 0.09h / dx^{0.5}$

= 0.3617

$T_{az} = 0.09h / dz^{0.5}$

= 0.4

Therefore ,C = 0.08 for medium soil

Seismic zoning factor for Kathmandu is, Z = 1.0

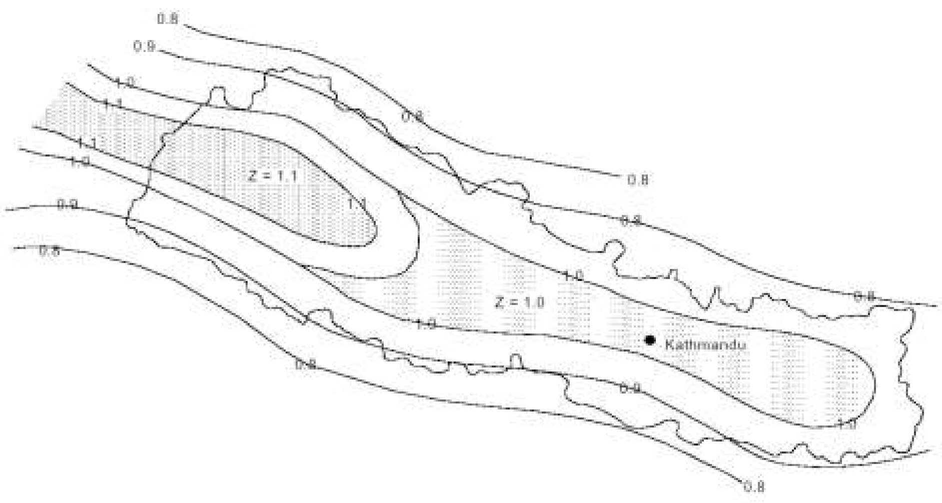


Figure 7-2 Seismic zone for Kathmandu

C_d = C Z I K

= 0.08 X 1 X 1 X 1

= 0.08

Base shear = Vb = C_d*W_t

= 3716.766 KN

7.1.4.3 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 (VB) computed in 1.5 shall be distributed along the height of the building as per the following expression:

$$F_i = V \times W_i h_i / \Sigma W_i h_i$$

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 hi (m)	Wi*hi	Wi*hi/∑Wihi	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

$\tau_{col} = (n_c/(n_c-n_f)) * (V_j / A_c)$< min of 0.4 Mpa and 0.1 sq.rt.(f_{ck})

$0.1\sqrt{fck} = \quad 0.45$

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	**nc**	**nf1**	**nf2**	**Ac**	**Storey Shears**	**Shear Stress**		**DCR**		**Remarks**
						τ colx (MPa)	**τ colz (MPa)**	**in x-dir**	**in z-dir**	
6	35	6	6	8.737	1007.52	0.14	0.14	0.35	0.35	**Since**

5	35	6	6	8.737	1812.96	0.25	0.25	0.63	0.63	**Demand Capacity Ratio is less than 1, hence Safe in shear**
4	35	6	6	8.823	2464.32	0.34	0.34	0.84	0.84	
3	35	6	6	9.593	2955.61	0.37	0.37	0.93	0.93	
2	35	6	6	10.80	3282.54	0.37	0.37	0.92	0.92	
1	70	8	8	17.12	3716.77	0.25	0.25	0.61	0.61	

Ʈcol< min of 0.4 MPa

Hence the check is satisfied

7.1.4.4 Axial Stress Check

Axial Stresses Due To Overturning Forces As Per Fema 310

a) 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

$F_o = 2/3\ (V_b/n_f) \times (H / L)$

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

$F_o = 2/3\ (V_b/n_f) \times (H / L)$

= 275.25 kN

Axial Stress for x-direction loading,

σ = 275.25 X 1000 = 1.72 MPa

0.16

σ_{all} =0.25 fck = 5.00 MPa

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

DCR = 0.334

b) 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

$F_o = 2/3 (V_b/n_f) \times (H / L)$

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 = 33.00 m

$F_o = 2/3 (V_b/n_f) \times (H / L)$

= 337.89 kN

Axial Stress for z-direction loading,

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

σ_{all} =0.25 fck = 5.00 MPa

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

DCR = 0.422

Hence the check is satisfied

7.1.4.5 Check For Out-Of-Plane Stability Of Brick Masonry Walls

Wall Type	Wall Thickness	Recommended Height/Thickness ratio (0.24 < Sx ≤ 0.35)	Actual Height/Thickness ratio in building	Comments
Wall in ground storey	230mm	18	(3800-450)/230 = 14.56	Pass
Wall in upper stories	230mm	16	(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.

7.1.5 DETAILED ANALYSIS

7.1.5.1 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 = 517kNm

f_{ck} = 20Mpa

f_y = 415 Mpa

Clear cover = 40mm

d′ = 40 + 10 + 25/2 = 62.5

d′/D = 0.104 ≈ 0.1

As = 4555.278 m^2

Percentage of reinforcement,

p = 1.265%

p/fck = 0.063

$P_u \backslash (f_{ck} bD)$ = 2572.51/(20 x 600 x 600)

= 0.0378

Referring to chart 44 of SP:16,

$M_u{'}/(f_{ck}\ b\ D^2) = 0.095$

$M_u{'}$ = 410.4 kNm

DCR = 1.259

Hence the check is not satisfied.

7.1.5.2 Shear Capacity Of Column

Considering that the steel in one face will be in tension,
As = 3 * π *25^2/4
= 1472.62 mm^2
Therefore, 100As/bd = 0.456
τ_c= 0.47 Mpa

Stirrups are 4- legged, 10mm Ø @ 200mm c/c spacing

Then,

Vus = 0.87 xfyxAsvx d/Sv

= 0.87 x 415x314.16 x 537.5/200

= 304kN

Therefore, Vu = Vus + $\tau_c$bd

= 456kN

Shear force per analysis = 332kN

Moment Capacity of Beam

$M_{u,lim}^{bR}$ = 194.27 kNm

$M_{u,lim}^{bL}$ = 497.67 kNm

h_{st}= 3.8 m

V from capacity design (IS13920)

= Vu = 1.4 ($M_{u,lim}^{bL}$ + $M_{u,lim}^{bR}$)/h_{st}

Hence, V_u = 254.925 kN

So, Final shear demand = 332 kN

V_u (=456kN) > Shear demand

DCR = 0.728

Hence, the check is satisfied.

7.1.5.3 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 mm^2

pt = 100 As/bd = 100x1257/(300x515) = 0.813%

Using table 19 of IS456:2000, for M20 grade of concrete and 100As/bd = 0.813,

τ_c= 0.575 MPa

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

$V_{us} = 0.87\ f_y\ .A_{sv}\ .\ d\ /\ S_v$

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

= (0.87 x 415 x 2 x78.57 x 515)/100 + 0.575 x 300 x 515 = 381.0 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 (M_R^H + M_R^S)/L_c

= 277.36 kN

Hence final shear demand in beam = 293.9kN

Vu (=381kN) > 293.9kN

DCR = 0.771

Hence, the check is satisfied.

7.1.5.4 Check For Strong Column Weak Beam

The flexure strengths of the columns shall satisfy the condition:

$\sum Mc \geq 1.1 \sum Mb$

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

The longitudinal beam of size 300 X 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 303.406kNm and 194.27 kNm respectively.

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

P_u / f_{ck}*b*D = 0.6625 where column size is 600mmX600mm

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

A_{sc} = 4555.278mm^2 ; pt = 1.265%

Therefore,

$M_u/f_{ck}*b*D^2$ = 0.01

M_u = 43.2kNm

$\sum Mc$ = 43.2+43.2 = 86.4kNm

$\sum Mb$ = 303.406 + 194.27 = 497.676kNm

1.1$\sum Mb$ = 547.437kNm

$\sum Mc <<< 1.1 \sum Mb$

Hence, check is not satisfied.

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

The longitudinal beam of size 300X550 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 265.3kNm and 153.1kNm respectively.

Factored column axial load = 2906.68kN

P_u / f_{ck}*b*D = 0.404 where column size is 600mmX600mm

The column is reinforced with 8-25dia.

A_{sc} = 3928.56mm^2 ; pt = 1.09%

Therefore,

$M_u/f_{ck}*b*D^2 = 0.065$

M_u = 280.8kNm

∑Mc = 280.8+280.8 = 561.6kNm

∑Mb = 265.3 + 153.1 = 418.4kNm

1.1∑Mb = 460.24kNm

∑Mc > 1.1 ∑Mb

Hence, check is satisfied.

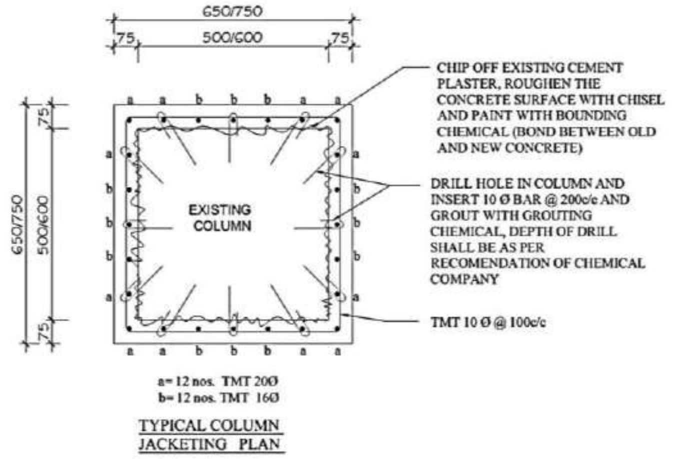
7.1.6 EVALUATION SUMMARY

- 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.

7.1.7 RETROFITTING OPTIONS

7.1.7.1 Option1: Rc Jacketing On Columns



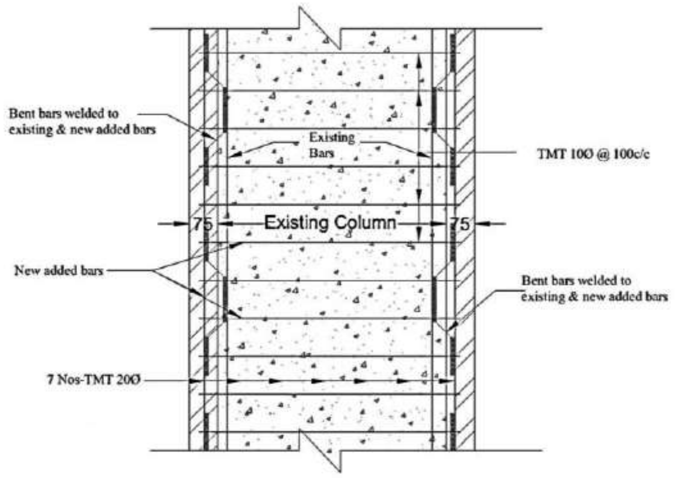


Figure 7-3 Colum Jacketing section

7.1.7.2 Option2: Steel Jacketing

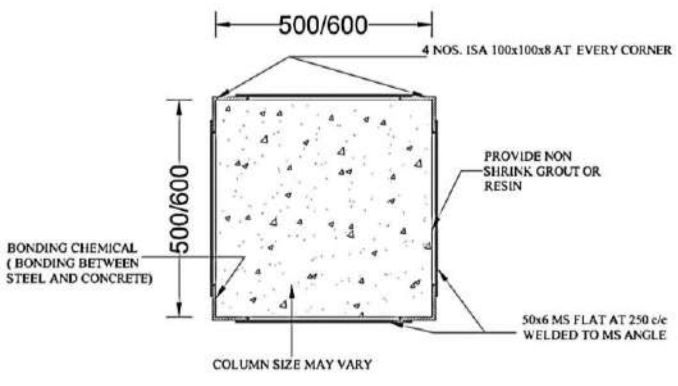


Figure 7-4 Typical column steel jacketing detail plan

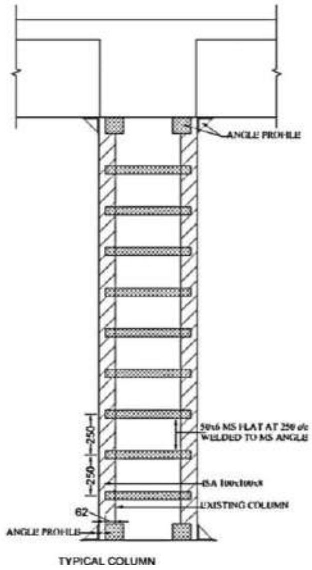


Figure 7-5 Steel jacketing detail elevation

7.1.7.3 Option3: Shear Wall Addition With Column Jacketing

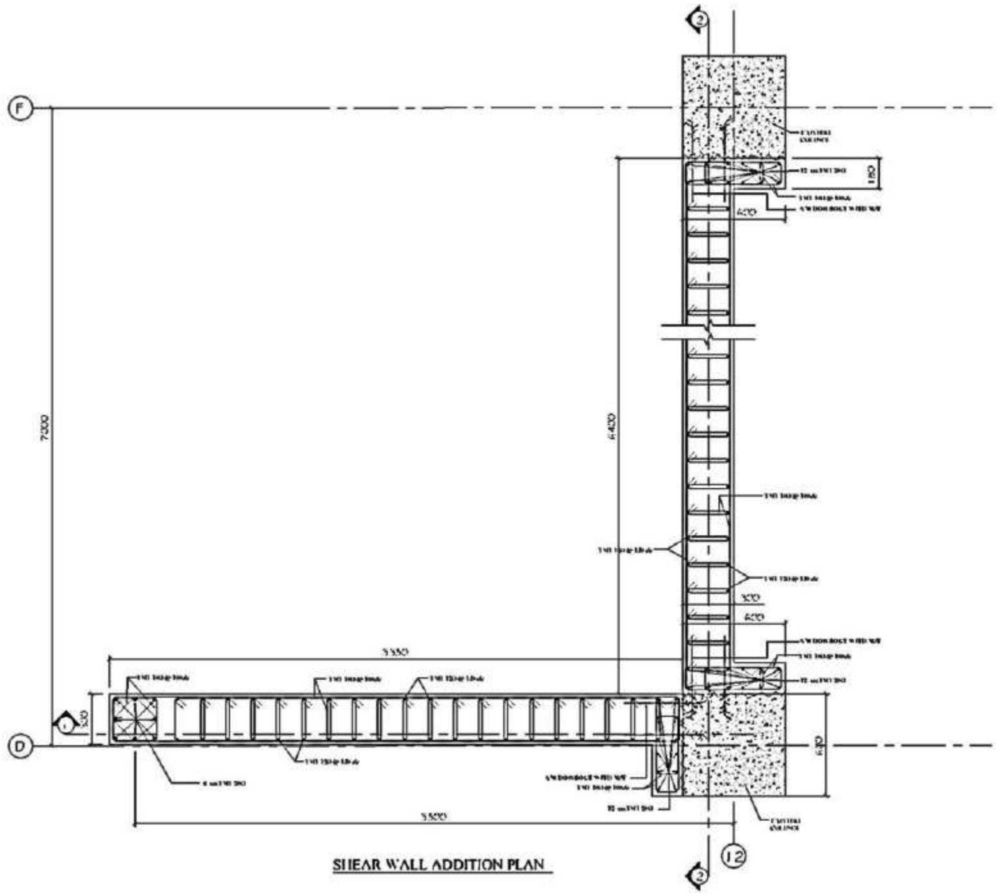
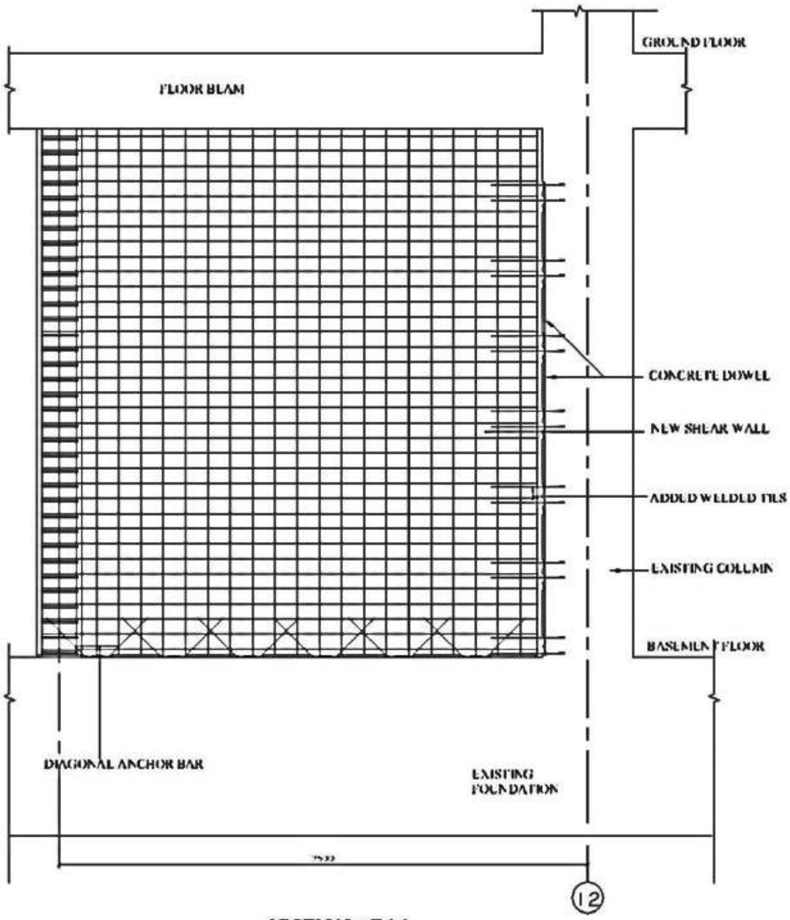
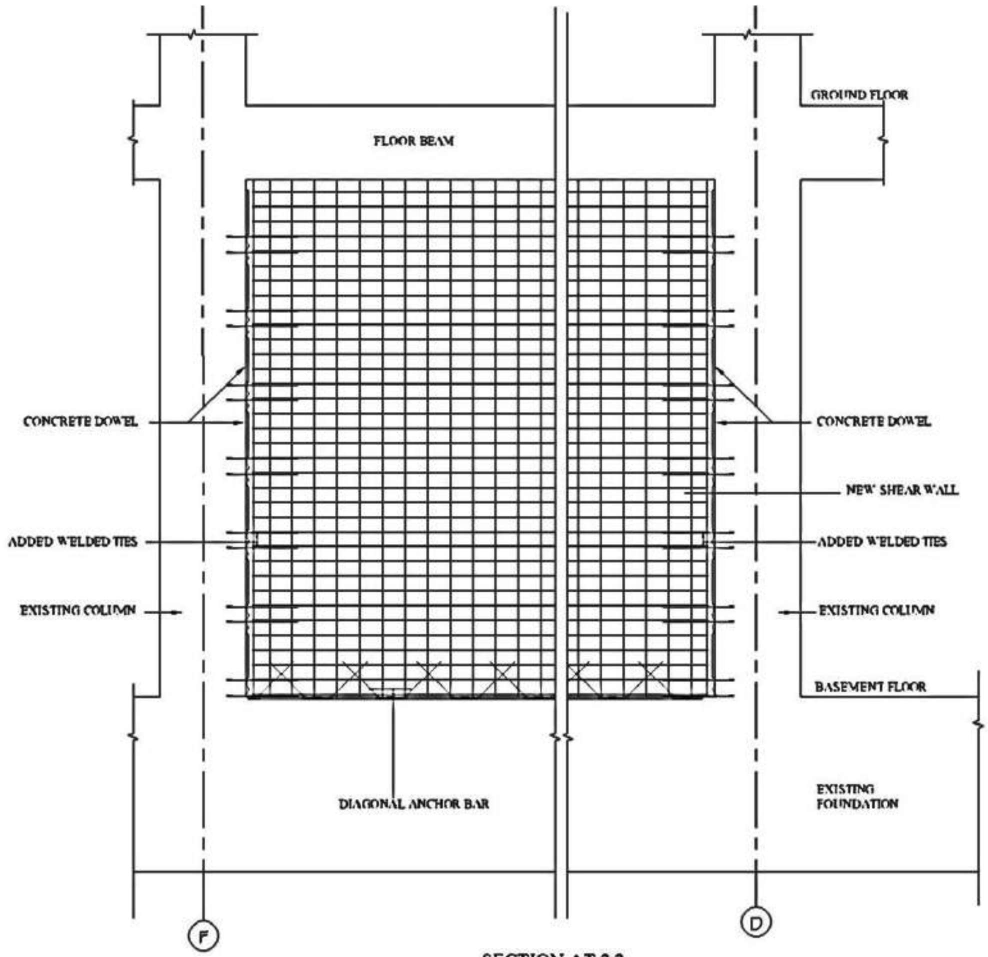


Figure 7-6 Steel wall addition plan



SECTION AT 1:1
(scale- 1.30)



SECTION AT 2.2
(scale- 1.30)

Figure 7-7 Sections

7.1.8 COST ESTIMATION OF RETROFITTING OPTIONS

- 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
- Shearwall 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

Alternative	Options	Parameter						
		Disturbance to existing structure	Time Consumption	Disturbance to existing function during construction	Cost	Damagibility after retrofitting	Effect on 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

7.1.9 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.

7.2 EXAMPLE 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 x 230mm, beam size of 230mm x 350mm, slab thickness of 125mm and storey height of 2.7m. It consists of 3- storey. The column size 230mm x 230mm is not sufficient referring to the latest Nepal Code which shows deficit at the site inspection itself.

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 to be converted into the health clinic.

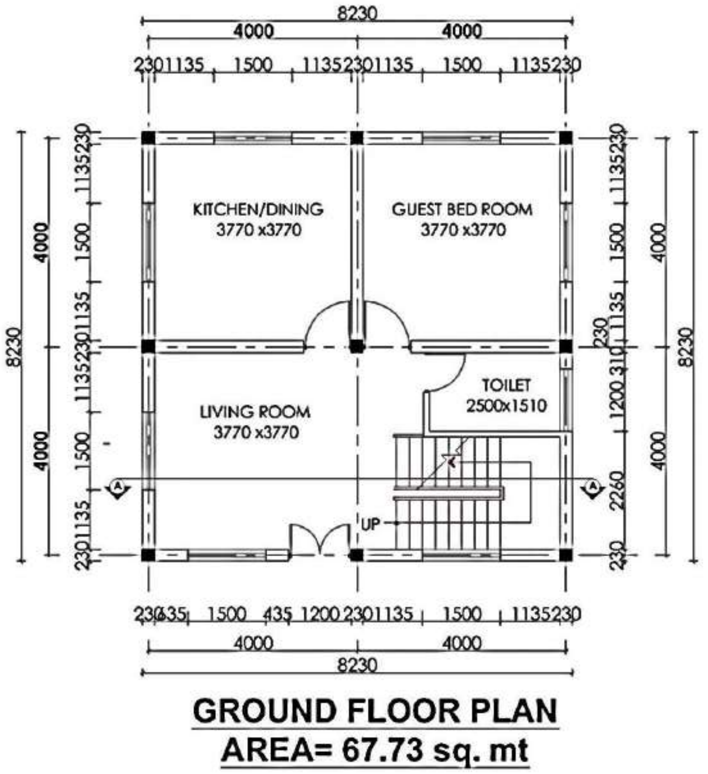
7.2.1 GENERAL DESCRIPTION OF EXISTING BUILDING

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 : 230mm *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

7.2.2 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.

7.2.3 BUILDING DRAWINGS



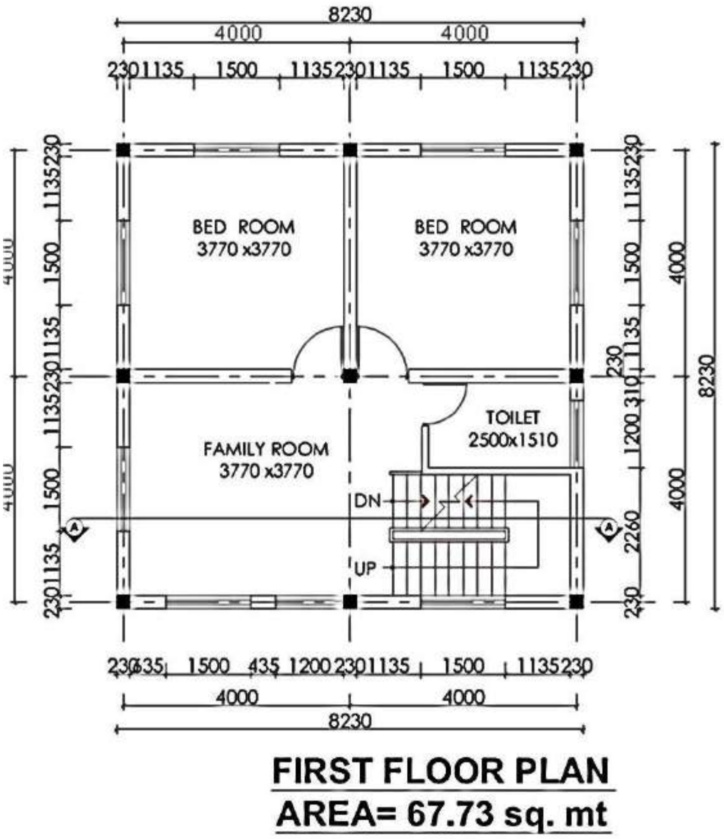
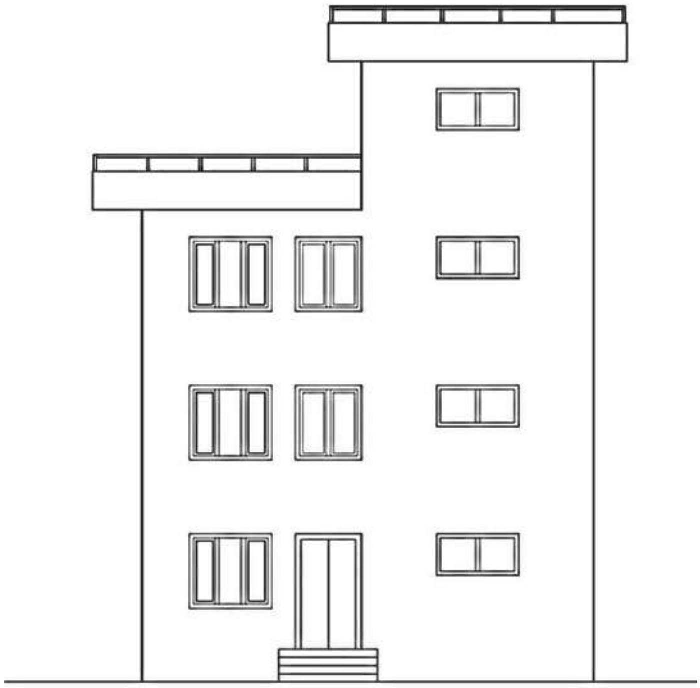
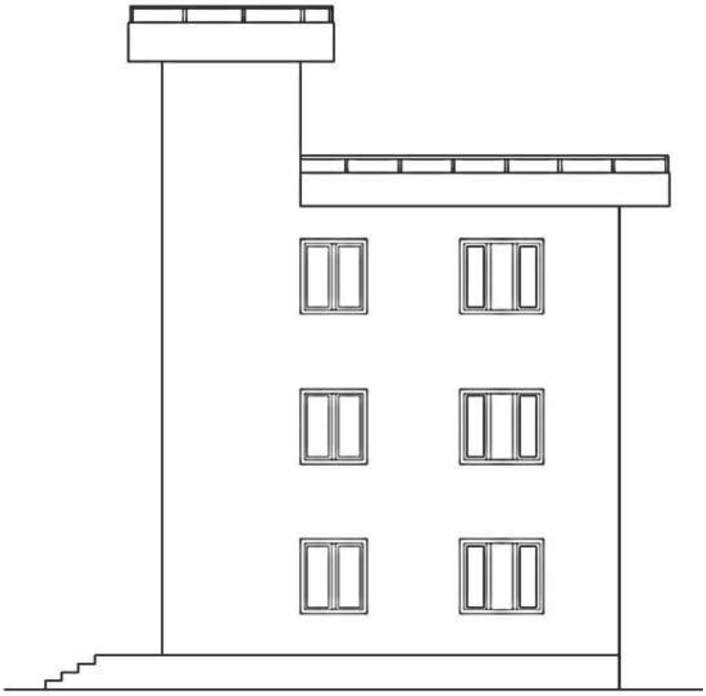


Figure 7-8 Building plan

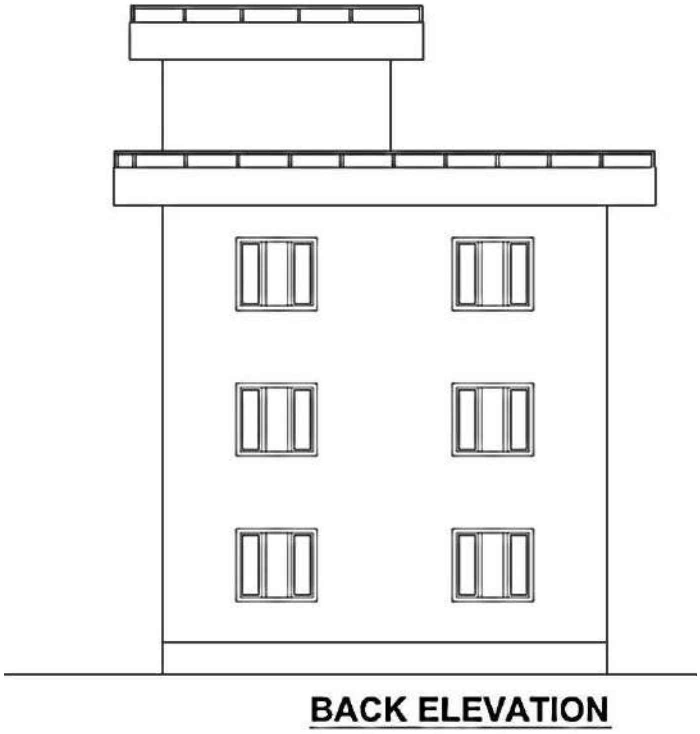


FRONT ELEVATION



SIDE ELEVATION

Figure 7-9 Front and side elevation



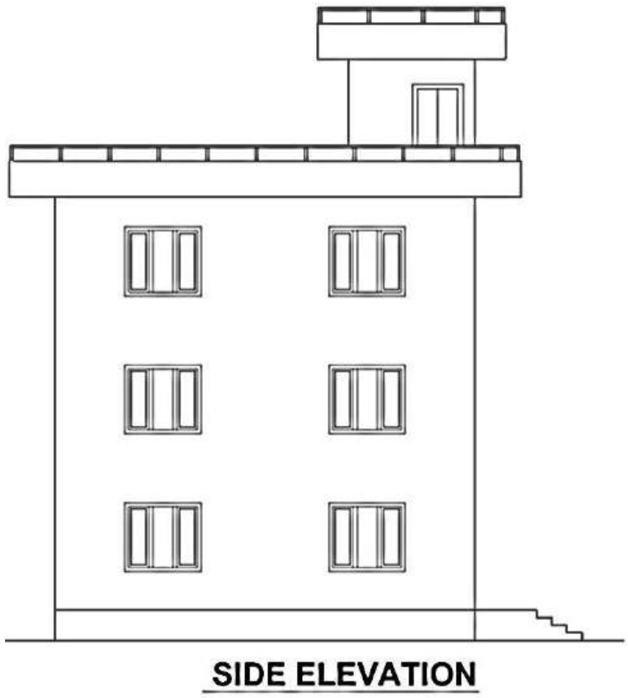


Figure 7-10 Back and side elevation

7.2.4 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)

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*)

7.2.5 LOAD CALCULATIONS

Dead Load :

1)For Different Floors:

Slab Load :	0.125 * 25 = 3.125 KN/m^2
Ceiling Plaster Load :	0.02 * 20 = 0.40 KN/m^2
Floor Finish Load :	0.025 * 20 = 0.50 KN/m^2
Marble Floor Load :	0.025 * 26.7 = 0.667 KN/m^2
Total Load =	4.692 KN/m^2

~ 4.70 KN/m^2

1) **For Roof Floor**:

Slab Load :	0.125 * 25 = 3.125 KN/ m^2
Ceiling Plaster Load :	0.02 * 20 = 0.40 KN/ m^2
Floor Finish Load :	0.025 * 20 = 0.50 KN/ m^2

Mosaic Floor Load : 0.025 * 20 = 0.50 KN/ m^2

Total Load : 4.525 KN/m2

~ 4.50 KN/m2

7.2.6 STRENGTH RELATED CHECKS

7.2.6.1 Analysis as Residential Building

The following is a detail of quick check calculations based on FEMA 310 for the seismic evaluation of building under consideration:-

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	FirstFloor	67.73	169.325	42.331	
1	Ground Floor	67.73	169.325	42.331	
∑				**152.18**	

b) LUMP MASS CALCULATION

S.NO.	FLOORS	Total Dead	Total Live	Total Weight	Remarks
		Load (KN)	Load (KN)	(KN)	

4	Third Floor	260.48	11.056875	271.54	
3	Second Floor	756.04	56.45625	812.50	
2	FirstFloor	649.82	42.33125	692.15	
1	Ground Floor	649.82	42.33125	692.15	
∑				**2468.34**	

c) CALCULATION OF BASE SHEAR

Calculation of base shear can be done using following codes:

a) IS1893:2002(Part 1)

b) NBC 105:1994

Based on IS 1893 (Part 1): 2002,

Calculation of earthquake loads using Seismic coefficient method:

The design horizontal seismic coefficient, $A_h = Z*I*S_a/2Rg$

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

$Ta = 0.09h / d^{0.5}$

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

Tax= 0.09h /dx0.5

=0.338 <0.55

Taz= 0.09h /dz0.5

=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 = 1.0 (For Residential 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) (Refer IS 1893 (Part 1) :2002-table 7)

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

A_h= ZISa/2Rg

= 0.36 * 1.0 *2.5/ 2 * 3

= 0.15

Base shear = Vb= Ah*W

=0.15* 2468.34

= 370.251 kN

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

Floor	Total weight	Height hi (m)	Wi*hi^2	Wi*hi^2/∑Wihi^2	Qi(KN)	Storey Shear Vi (KN)
	Wi (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
∑	**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,

Vb=Ah x W

Average Shearing stress in columns is

given as

Tcol = (nc/(nc-nf)) * (Vj / Ac)< min of 0.4 Mpa and 0.1 sq.rt.(fck)

For Ground Storey columns,

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

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

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

Vj = Maximum Storey Shear at storey level 'j'

Shear Stress at Storey Levels

Storey	nc	nf1	nf2	Ac	Storey Shears	Shear Stress	
					(KN)	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

Tcol>>min of 0.4MPa and 0.1sqrt(fck) = 0.45 MPa

Hence, the check is not satisfied.

7.2.6.2 Analysis as Health Clinic :

Major changes while converting Residential building into Health clinic

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

The following is a detail of quick check calculations based on FEMA 310 for the 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	LL	0.25LL	Remarks
		sq.m			
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**	

b) LUMP MASS CALCULATION

S.NO.	FLOORS	Total Dead	Total Live	Total Weight	Remarks
		Load (KN)	Load (KN)	(KN)	
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	
∑				**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, $A_h = Z*I*S_a/2Rg$

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 = 0.09h /d0.5

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

Tax = 0.09h /dx0.5

=0.338 <0.55

Taz = 0.09h /dz0.5

=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)(Refer IS 1893 (Part 1) :2002-table 7)

Ah = ZISa/2Rg

= 0.36 * 1.5 *2.5/ 2 * 3

= 0.225

Base shear = Vb = Ah*W

= 0.225* 2496.56 = 561.726 KN

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

Floor	Total weight	Height hi (m)	$Wi*hi^2$	$Wi*hi^2/\sum Wihi^2$	Qi(KN)	Storey Shear Vi (KN)
	Wi (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
∑	**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

Vb=Ah x W

Average Shearing stress in columns is given as

Tcol = (nc/(nc-nf)) * (Vj / Ac)< min of 0.4 Mpa and 0.1 sq.rt.(fck)

For Ground Storey columns,

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

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

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

Vj = Maximum Storey Shear at storey level 'j'

Shear Stress at Storey Levels

Storey	nc	nf1	nf2	Ac	Storey Shears	Shear Stress	
					(KN)	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

Tcol>>min of 0.4 MPa and 0.1sqrt(fck) = 0.45 MPa

Hence, the check is not satisfied.

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

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

- ***Checking Shear Capacity of Center Column :***

Shear Capacity of column required = 1.4 (M1+ m1')/ hst

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 -25-16/2

= 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 :

Vu = 1.4 (M1+ m1')/ hst

= 1.4*(68.6+53.6)/2.7

= 63.36 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

for Pt = 0.85% ,τ = *0.585 N/mm*^{2}

Shear Capacity = 0.585*230*230/1000

= 30.94 kN

Shear to be carried Stirrups Vus = 63.36 – 30.94

= 32.42 KN

From SP 16 Table 62 :

Stirrups in the Column : Tor 8 Diameter @150 mm c/c

Vus/ d = 2.42 kN/ cm

Vus= 2.42 * 19.2 kN/ cm

= 46.5 kN>>32.42 KN

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

g) Axial Stress Check:

The Axial Stress due to Gravity Loads as per FEMA 310

Permissible axial stress = 0.1fc' = 2 N/mm^2

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 = 711.289*1000/230/230

= 13.446 N/mm^2 > 2 N/mm^2

Hence the check not satisfied.

***h)* Axial stresses due to overturning forces as per FEMA 310**

<u>Axial stress in moment frames for x-direction loading</u>

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

$F_o = 2/3\ (V_b/n_f) \times (H / L)$

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

$F_o = 2/3\ (V_b/n_f) \times (H / L)$

= 252.78 kN

Axial Stress for x-direction loading,

$$\sigma = \frac{252.78}{0.05} = 4.78 \text{ MPa}$$

σ_{all} =0.25 fck = 5.00 MPa

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

DCR = 0.334

Hence the check is satisfied.

<u>Axial stress in moment frames for z-direction loading</u>

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

$F_o = 2/3\ (V_b/n_f) \times (H / L)$

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

$F_o = 2/3\ (V_b/n_f) \times (H / L)$

= 252.78 kN

Axial Stress for x-direction loading,

$$\sigma = \frac{252.78}{0.05} = 4.78 \text{ MPa}$$

σ_{all} =0.25 fck = 5.00 MPa

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

DCR = 0.334

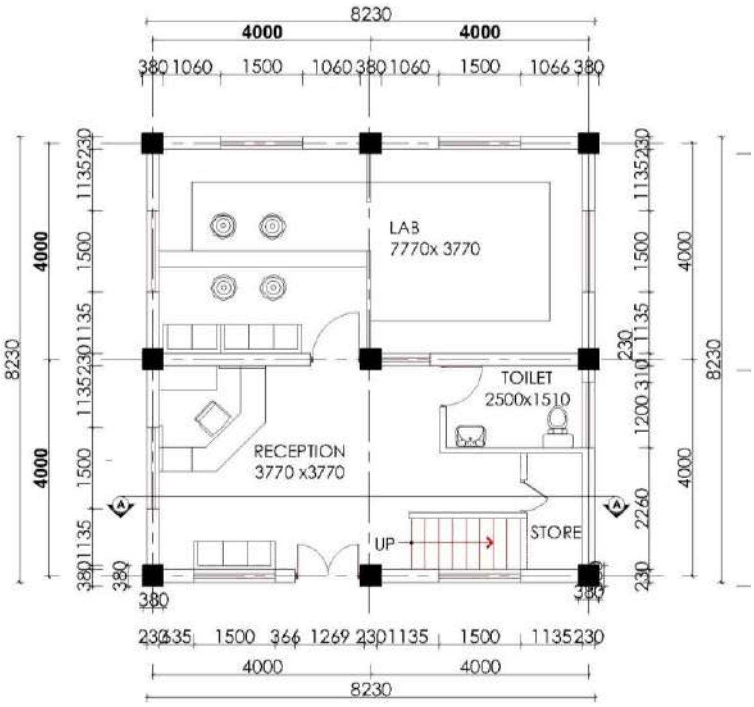
Hence the check is satisfied

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

Wall Type	Wall Thickness	Recommended Height/Thickness ratio (0.24 <Sx ≤ 0.35)	Actual Height/Thickness ratio in building	Comments
Wall in ground storey	230mm	18	(2700-350)/230 = 10.217	Pass
Wall in upper stories	230mm	16	(2700-350)/230 = 10.217	Pass

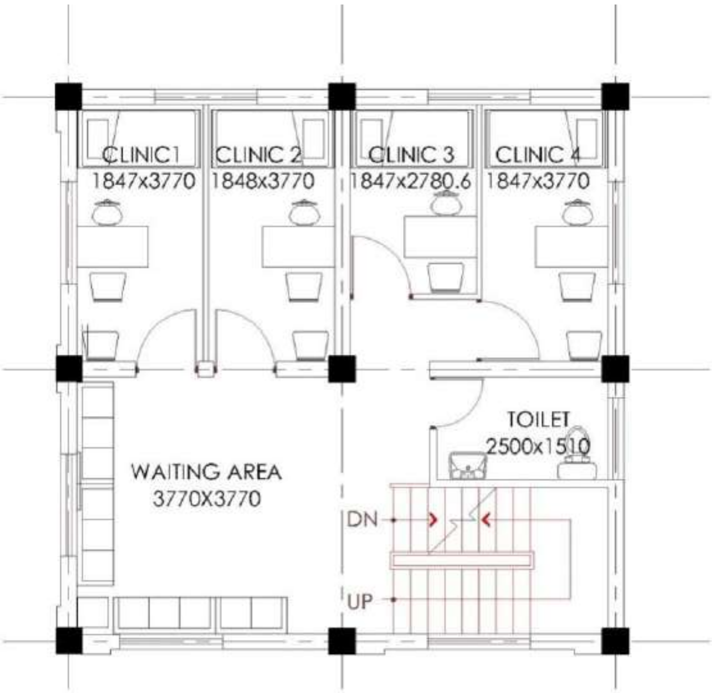
Hence the check is satisfied.

7.2.7 RETROFITTING DRAWINGS

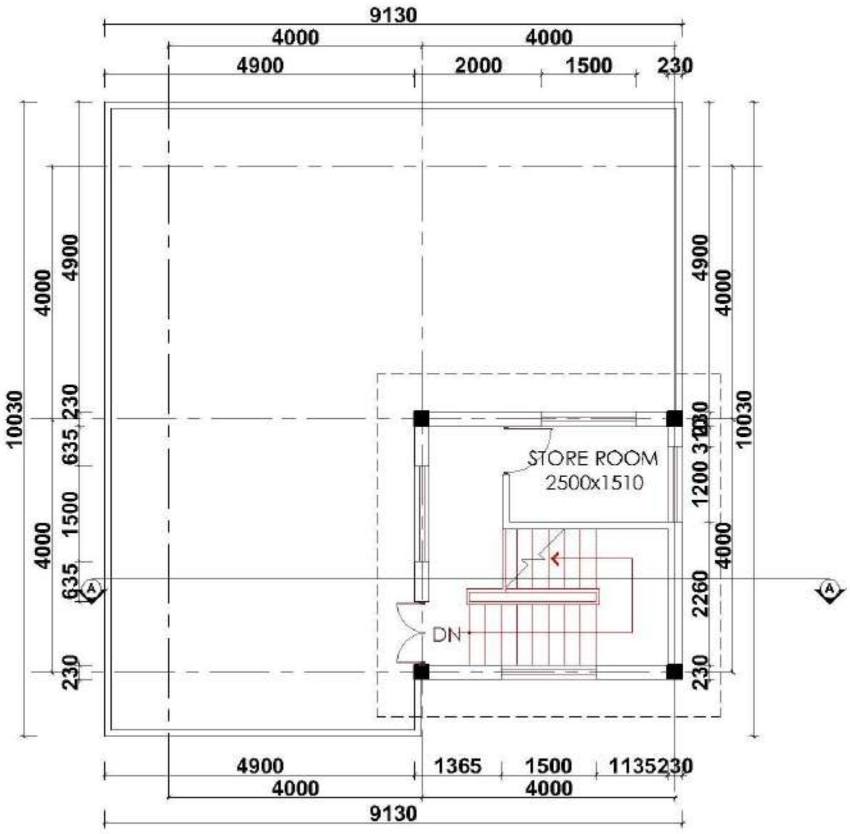


GROUND FLOOR PLAN
AREA= 67.73 sq. mt

Figure 7-11 Retrofitted ground floor plan

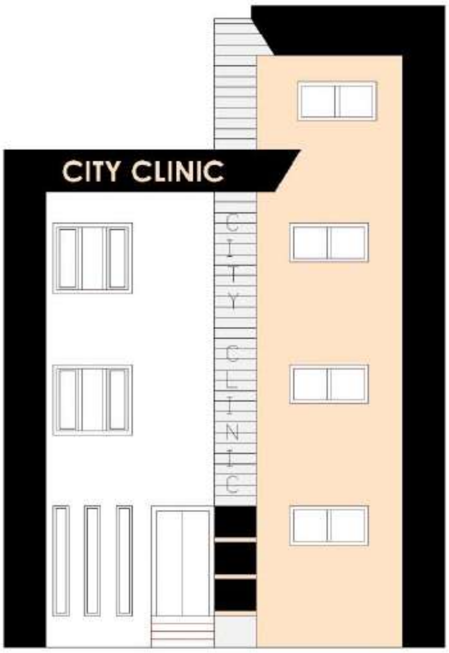


FIRST FLOOR PLAN
AREA= 67.73 sq. mt

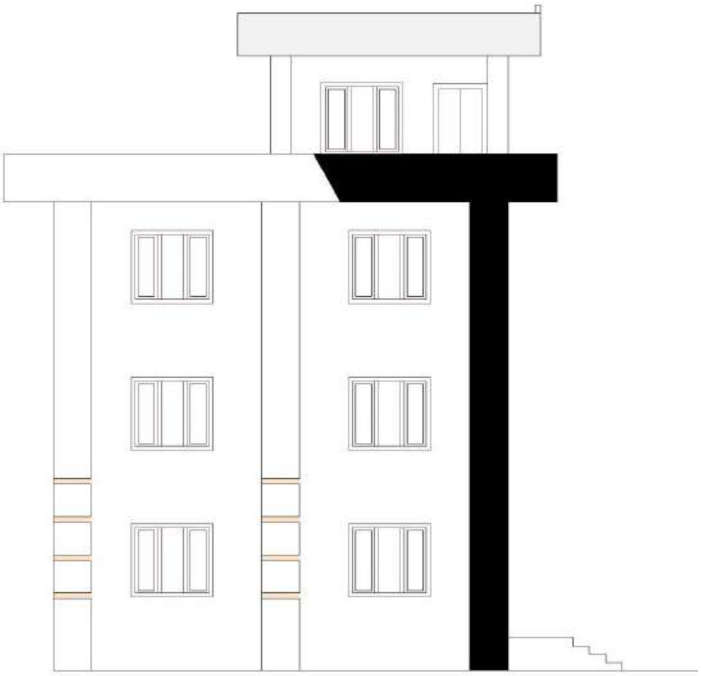


TOP FLOOR PLAN

Figure 7-12 Retrofitted first and top floor plan



FRONT ELEVATION



SIDE ELEVATION

Figure 7-13 Front and side elevation

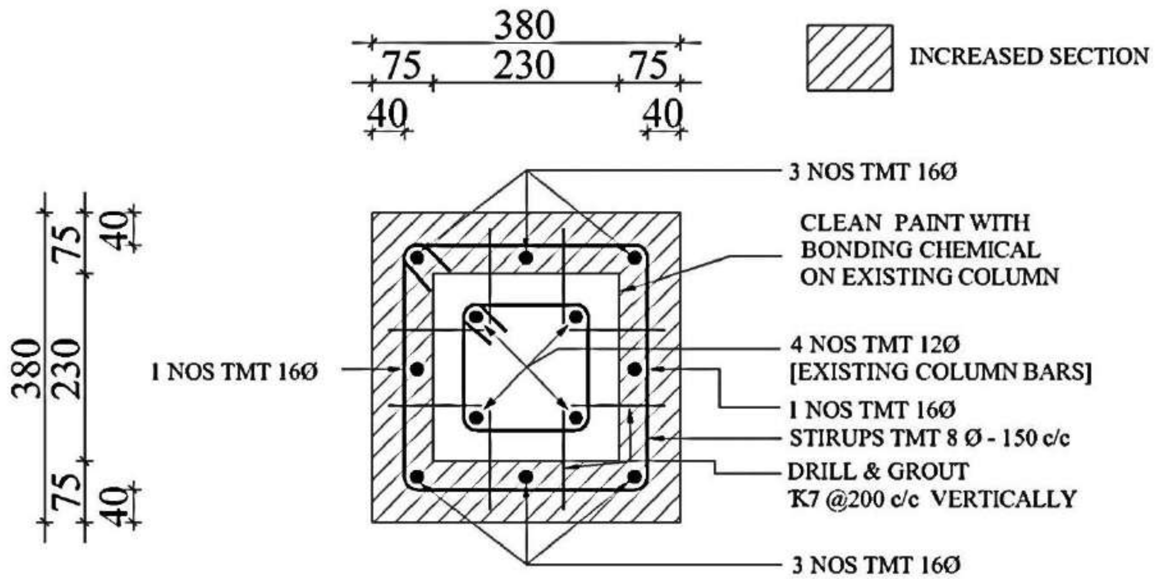


Figure 7-14 Section of jacketed column C1

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