Earthquake Resistant Foundation System of Low Rise Building

by

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NOTATIONS

Q_{ult} = Ultimate bearing capacity of soil

q_a = Allowable bearing capacity of soil

C_c = Compression index

e_o = Initial void ratio

E_m = The modulus of elasticity of the infill material

f'_m = Compressive Strength of Masonry Wall

K_i = Stiffness of infill wall

Ic = The moment of inertia of column

t = The thickness of infill wall

T = Fundamental period

P_A = Wall density

V_{max} = Maximum shear force

URM = Unreinforced masonry

PGA = Peak ground acceleration

g = Acceleration due to gravity

SDC = Seismic Design Category

CM = Confined Masonry

μ = Frictional coefficient

K_s = Modulus of subgrade reaction

ΔH = Foundation Settlement

δ = Deflection

Q = Storey shear force

M_w = Magnitude of earthquake

ν = Poisson's ratio,

α_t = Coefficient of Thermal expansion,

G = Shear Modulus

f'_c = Compressive Strength of Concrete at 28 days

f_y = Yield Stress of Reinforcement,

T = Fundamental period

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ABSTRACT

Most low rise buildings in Bangladesh are reinforced concrete frame with unreinforced solid clay brick infill wall as a partition. The frames are designed as the primary lateral force resisting system; the stiffness and strength of brick infill walls are not conventionally considered in analysis and design. So isolated column footing is commonly provided in low rise buildings.

Infill brick wall significantly affect the response of RC frame structure during earthquake. The change in lateral stiffness, strength and natural period of the frame structure due to the presence of infills change the behavior of the building under seismic action.

Therefore, continuous wall footing for a low rise building can be an earthquake resistant foundation system because of (i) infill wall between foundation and grade beam (ii) symmetry in elevation (iii) uniformity and continuity in horizontal and vertical direction and (iv) integral action of foundation and super-structure.

To investigate the seismic resistant behavior, a 3-storied frame structured building was studied under conventional method and confined method. In conventional method brick wall is considered as loads on beams and slab. Stiffness and strength is completely ignored in analysis and design. So conventional method did not shows real behavior of the buildings during earthquake. In confined method masonry brick wall was modeled as structural element. Compressive strength and stiffness of masonry wall was considered in analysis and design. Also confined masonry construction method was adopted for construction which based on “first brick wall and then frame”.

Based on this study it was found that confined continuous masonry wall footing is the better seismic resistant foundation than conventionally designed isolated column footing for low rise buildings. Also consideration of infill brick wall in analysis and design of RC frame structured building and adopting confined method of construction increases lateral resistance, storey shear capacity, lateral stiffness, energy dissipation

capacity, improve ductility behavior and decreases storey displacement of the structure rather than conventional bare frame method of analysis and design. Performance of confined masonry building and conventional bare frame building in several part earthquake already proof these benefits. Also confined method reduces the foundation cost approximately by 21.5% and total cost of the building by 15% rather than conventional method of low rise building. Finally it is concluded that the confined method of analysis, design and construction of RC frame structured low rise building is more seismic resistant than conventional bare frame method.

CHAPTER 1
INTRODUCTION

1.1 General

Masonry construction was the leading structural system for low and medium rise buildings until the beginning of the twentieth century. The effective use of masonry construction is seen in load bearing structures where it performs a variety of functions such as supporting loads, dividing space, providing thermal insulation and weather protection. But its lateral load resisting capacity is so much smaller, that is why masonry buildings collapse regularly during earthquake. Later on it was replaced by new construction materials viz. reinforced concrete and structural steel. However it is still in use particularly for low cost housing and low rise building, but mostly for partition purpose.

Masonry infill in RC frames acts as a diaphragm in vertical plane that imparts significant lateral strength and stiffness to RC frames under lateral loads. Infilled frames also tend to be substantially stronger, but less deformable, than otherwise identical bare frames. In symmetric buildings with vertically continuous infilled frames, the increased stiffness and strength may protect a building from damage associated with excessive lateral drift or inadequate strength. Because of its higher stiffness, infill panels may attract significantly greater forces that may lead to premature failure of infill, and possibly of the whole structure. Also existence of infill wall in real structure significantly changes the response of the foundation and RC frame structure. Therefore, it is essential for designers to consider the effects of infills in the design of RC buildings.

The basic principle of any design is that the product should meet three basic requirements namely, (i) function, (ii) cost, and (iii) reliability. While the terms function and cost are simple in principle, reliability concerns various technical factors relating to serviceability and safety. As the above three criteria are interrelated, and because of the normal constraints on cost, compromise with function and reliability generally have to be made. In considering the means of achieving the above requirements it is necessary to take into account both the limitations and the

opportunities arising from the availability of construction materials and components and of construction skills. In seeking the optimum of the proposed construction, designers should choose forms and materials that give the best failure modes in earthquakes with functional and cost requirements. The form or configuration of the construction is the geometrical arrangement of all of the elements, i.e., structure, architecture, equipment, and contents. In order to achieve reliable earthquake resistance the form of construction should be decided from consideration of the following factors: (1) Simplicity and symmetry, (2) Length of plan, (3) Shape in elevation, (4) Uniformity and continuity, (5) Stiffness, (6) Failure mode, and (7) Foundation conditions. Earthquake repeatedly demonstrated that the simplest structures have the greatest chance of survival. Symmetry is desirable for much the same reasons. It is worth pointing out that symmetry is important in both directions in plan and in elevation. Lack of symmetry produces torsional effects which are sometimes difficult to assess and can be very destructive.

The basic rule regarding the earthquake resistance of substructure is that integral action in earthquakes should be obtained. This requires adequate consideration of the dynamic response characteristics of the superstructure and of the subsoil. If a good seismic-resistant form has been chosen for the superstructure then at least the plan form of the substructure is likely to be sound, i.e. (1) Vertical loading will be symmetrical, (2) overturning effects will not be too large, and (3) The structure will not be too long in plan. In earthquake areas seismic design will involve consideration of the following factors: (a) Transmission of horizontal base shears from the structure to the soil, (b) Provision for earthquake overturning moments (e.g. tension piles), (c) Differential settlements, (d) Liquefaction of the subsoil, and (e) The effect of embedment on seismic response. The effects of depth of embedment are not fully understood at present, but some allowance for this effect can be made in soil structure interaction analyses, or when determining at what level to apply the earthquake loading input for the superstructure analysis.

1.2 Background of the Study

The behavior of masonry in-filled frame structures has been studied in the last four decades in attempts to develop a rational approach for design of such frames. Present code of practice of Bangladesh (BNBC-1993) does not include provision of taking into consideration the effect of infill. Masonry infill walls are widely used as Partitions worldwide. Previous field evidence has shown that continuous infill masonry wall can help to reduce the vulnerability of reinforced concrete Structure.

RC Frames with unreinforced masonry infill walls are common in developing countries like Bangladesh. Often engineers do not consider masonry infill walls in the design process because the final distribution of these elements may be unknown to them or masonry walls are regarded as non-Structural elements or short of skill of engineers to counting the effect of this walls or the risk due to the tendency of design violation of our country's land owner for beautification of room by glass/others. But walls and frames interact during ground motion at the same time. This leads to Structural response deviating radically from what is expected in the design. Previous experimental research on the response of RC frames with masonry infill walls subject to static and dynamic lateral cyclic loads (Broken and Bertero, 1981; Calvi and Bolognini, 2001; Hashemi and Mosalam, 2006) have shown that infill walls lead to significant increase in strength and stiffness in relation to bare RC frames. This increase in stiffness is beneficial because they lead to reductions in the magnitude of the deformations induced by ground motions. "Masonry walls can be used to reduce the vulnerability of RC frames" has been proved by testing full scale low rise RC structure without infill walls & with infill walls (Pardo et al, 2008).

Now-a-days in rural areas of Bangladesh 1-3 storied RC frame structure buildings are constructed by local masons where individual shallow footings are not properly designed to resist differential settlement and to resist earthquake load. Moreover earthquake resistance of RC frame structure depends on not only proper design but also proper detailing and construction method. Also earthquake resistance of foundation also depends on the tie action at the base level to resist the base shear. It is necessary to develop an economic and more earthquake resistant foundation system

for low rise buildings which can be built by local masons without difficulties. Benefit of infill wall can be utilized to develop such economic foundation system. In this project a foundation system is proposed which utilizes the benefit of infill masonry wall.

An infill wall constructed after construction of RC frames and an infill wall constructed as an integrated part of RC frames will behave differently. This effect of construction sequence was considered in the proposed foundation system. In the proposed foundation system, RC wall footing will be constructed first; infill masonry wall will be constructed on RC wall footing keeping space for columns. When masonry wall will reach up to the bottom of the grade beam, the columns will be constructed. Then grade beams will be constructed directly on masonry infill wall and columns. This construction sequence may be called “infill wall first; then RC frames”. The wall footing, columns and grade beams will act as integrated or composite structure to resist the lateral loads induced by earthquake. Thus the foundation system would be earthquake resistant.

1.3 Objectives of the Study

The objectives of the study are as follows:

i) To determine the effect of infill masonry walls in the proposed foundation system of low rise building.

ii) To determine the seismic resistance capacity of foundation and superstructure of low rise building.

iii) To study the behavior of RC frame with brick masonry infill by modeling masonry infill as continuum infill panel i.e. finite element model using package ETABS.

iv) To prepare guidelines for confined masonry construction (first brick work, then frame) and spread it to the construction industry.

v) To compare the cost of proposed foundation system and conventional foundation system.

1.4 Methodology

In this study a 3-storied frame structured building was modelled using finite element package ETABS considering conventional foundation system and proposed foundation system. In the conventional system the walls was considered as load on beams whereas in the proposed foundation system the wall was considered as structural element. Since the brick masonry walls have only compressive strength and stiffness, i.e. they cannot resist bending in any direction, bending stiffness of masonry wall was made as near to zero in ETABS model in proposed confined method. Gravity loads and earthquake loads will be considered to design the building. In confined method, nonlinear finite element model was constructed by direct frame elements and masonry wall i.e. continuum idealization for the infill panel and frame (Taher and Afefy, 2008) where effect of interface material was modeled by gap elements which has compression stiffness but no tensile stiffness. The infill panel is macro mechanically treated as homogeneous material and the effect of mortar joints between masonry units is smeared over the whole panel and taken on an average sense (Haddad, 1991) and so on an average compressive strength & moduli is taken for analysis. Considering foundation system was integrated with frame structure (as a whole rigid body) in confined method and foundation was continuous masonry wall footing having contact with RCC base to RCC grade beam. Foundation of confined method was modeled by line spring under a beam on the basis of beams on elastic foundation where modulus of subgrade reaction or spring stiffness was calculated for a factor of safety 3.0 (Bowles, 1997). In conventional model, foundation was taken as individual column footing and modeled as hinge. Based on the results of finite element structural analysis of ETABS, the building was designed considering conventional foundation system and proposed confined foundation system as per BNBC-1993. Seismic resistant capacity i.e. foundation settlements, lateral stiffness, ductility, energy dissipation capacity, lateral resistance, storey shear capacity and deformability of both the foundation as well as whole structure was evaluated by various numerical measurement index stated in the code. Verification of seismic capacity was referred from some most recent research work. Performance of the confined and conventional building in a few recent earthquakes also included in this

project work for real life example. Also material estimate and BOQ of both the system of design and construction was performed to compare cost of the conventional system and confined system.

1.5 Organization of the Thesis

The thesis is arranged into five chapters and one appendix. In Chapter One, background, aims and objectives and method of investigation of the research is described. Chapter Two contains the literature review where history, use and researches on earthquake resistant foundation for low rise building, confined masonry construction and conventional bare frame method are described. In this chapter description of details analysis and design consideration for earthquake resistant foundation, response of RC structures, performance of confined infill wall, modeling techniques of infill, inclusion of infill's influence in current code are given.

Chapter Three describe the numerical finite element modeling of infill wall in continuous wall footing and in the whole structure, finite element modeling of RC frame members. Chapter Four contains results and discussion. Chapter Five contains the conclusions and recommendations for further research. Architectural and structural design drawing are presented in Appendix A. Details estimate of materials, BOQ are presented in Appendix B and wall density calculation is presented in Appendix C.

CHAPTER 2
LITERATURE REVIEW

2.1 General

Earthquake or seismic performance defines a structure's ability to sustain its due functions, such as its safety and serviceability, at and after a particular earthquake exposure. A structure is normally considered safe if it does not endanger the lives and well-being of those in or around it by partially or completely collapsing. A structure may be considered serviceable if it is able to fulfill its operational functions for which it was designed. Low rise building foundations are typically called light foundations, which can be founded on expansive soils, clay or sand. Low-rise buildings are sometimes defined as one to four stories in height. These buildings include houses, garages, apartment and condominium buildings, restaurants, schools, mosques, and other similar structures. Design options for foundation systems are foundation components, and moisture and vegetation control methods. There is no absolute design rules for choosing a type of foundation design for a building, but it has to design based on its performance. This project describes the guideline to select the best seismic resistant foundation system for low rise frame structured building. If the soil under the house moves up, down, or sideways, the house is in trouble. Designers of homes may know quite a bit about the soil conditions at a site and may design the building exactly for those conditions. More commonly, soil conditions are assumed to be a certain type, and footings and foundations are designed with a margin of safety to account for adverse soil conditions, within reasonable limits.

2.2 General Foundation Design Considerations

Aside from supporting the building loads, the goal of structural foundation design in clay or expansive soil areas should be to economically mitigate the detrimental effects of foundation. Foundation elements must be proportioned both to interface with the soil at a safe stress level and to limit settlements to an acceptable amount. Foundation movements can be reduced by providing tie action at the base level. The tie action can be provided by combined footing design, strap footing design or mat design. But all these are very costly. For low rise building supporting expansive clay, it is required an

economic and safer foundation surviving earthquake. So an economic tie action can be achieved by providing continuous wall footing and it can be make better earthquake resistant by following confined method.

Another general design consideration is the effect of the magnitude of surcharge pressure on the degree of swell that can occur. Lightly loaded foundation components, such as concrete flatwork, pavements, and building slab-on-grade floors, are impacted more by expansive soil volumetric changes than are heavily loaded foundation components such as heavily loaded bearing walls. Heavy loads reduce the amount of swell than can occur.

The designer is responsible for selecting the appropriate foundation. The following factors can also be considered during foundation design:

i) Design load: The magnitude of the design load dictates the required size of the foundation from a structural standpoint.
ii) Subsurface formations: The depth and strength of subsurface formations determine the type of foundation chosen. In general, for low depth foundation spread or combined footing can be choose for light structure. While piling is required for very compressible soil or having tendency to liquefaction. In general, use piling where softer soil is present. Depth of foundation must be well below the zones of freezing, thawing and plant growth, peat and muck, organic materials.
iii) Corrosive conditions: Salts, chlorides, and sulfates are detrimental to foundations. Where these conditions exist, preventive measures are required. Sulfate-resistant concrete can be use for construction in seawater or high sulfate known areas of possible corrosion due to sulfate soil or salt water.
iv) Economic considerations: Economics is considered in the final selection comparing the foundation types. The cost of a continuous wall footing, for instance, obviously is less than isolated or combined footing for low rise frame structured building where earthquake load is considered.
v) Superstructure type: The type of superstructure chosen for the buildings may dictate or eliminate certain foundation types. For instance, uniform span low rise structures i.e. moment resisting frames may work well with wall footing, but long span and tall building may required MAT foundation or piling.

vi) Construction Method: Construction with locally available manpower or materials is a governing factor for foundation design. So for rural and semi urban areas where local masons construct the foundation, wall footing is best suited than isolated footing for easy of construction.

vii) Special design requirements: Special designs are sometimes necessary to straddle another structure or utilities and may require a different type of foundation than the rest of the structure.

Numerous foundation system design options are available that meet these goals to varying degrees. Many options are also available in the design and selection of components that make up these foundation systems; however, choices should be based upon an engineered geotechnical investigation. Different options are also available in the design of the site around the foundation and the selection of landscaping components.

2.3 Foundation System of Low rise building

This section discusses the various types of foundation systems that are commonly used for residential and other low-rise buildings. In this document, the foundation system is considered to include the structural floor framing system at or near grade level and all other structural components beneath the building. The building superstructure consists of all structural elements above the grade level floor.

The foundation systems are subdivided into two groups: deep support systems and shallow support systems. Each of these systems has an associated level of risk of damage that can occur to the building superstructure and architectural components due to differential foundation movements. Each of these systems also has an associated relative cost of construction. When comparing the various foundation systems, the level of risk is typically found to be inversely proportional to the level of cost. Higher risks are often accepted due to economic considerations. For example, shallow support systems typically have a relatively higher level of risk than deep support systems, but are often selected due to economics and affordability. Because risk of damage and economic considerations are involved, building owners and/or developers need to be involved in the selection process of the foundation system.

Shallow foundations are those founded near to the finished ground surface; generally where the founding depth is less than the width of the footing. If surface loading or other surface conditions will affect the bearing capacity of a foundation it is 'shallow'. Shallows foundations are used when surface soils are sufficiently strong and stiff to support the imposed loads; they are generally unsuitable in weak or highly compressible soils, such as poorly-compacted fill, peat, recent alluvial deposits, etc. Two common types of shallow foundation for low rise buildings are wall footing and isolated column footing. But conventionally isolated column footing is frequently designed for low rise building due to less time required for modeling and design. Conventionally only hinge support is attached at base level. But earthquake resistant capacity has to ensure for any kind of footing to save man and money by proper modeling, analysis and design.

Isolated column foundations or pad foundation are used to support an individual point load such as that due to a structural column. They may be circular, square or rectangular. They usually consist of a block or slab of uniform thickness, but they may be stepped or haunches if they are required to spread the load from a heavy column. Masonry wall footing or strip foundations are used to support a line of loads, either due to a load-bearing wall, or if a line of columns need supporting where column positions are so close that individual pad foundations would be inappropriate. Masonry wall footing consists of RCC base and masonry wall continuously provided below grade beams.

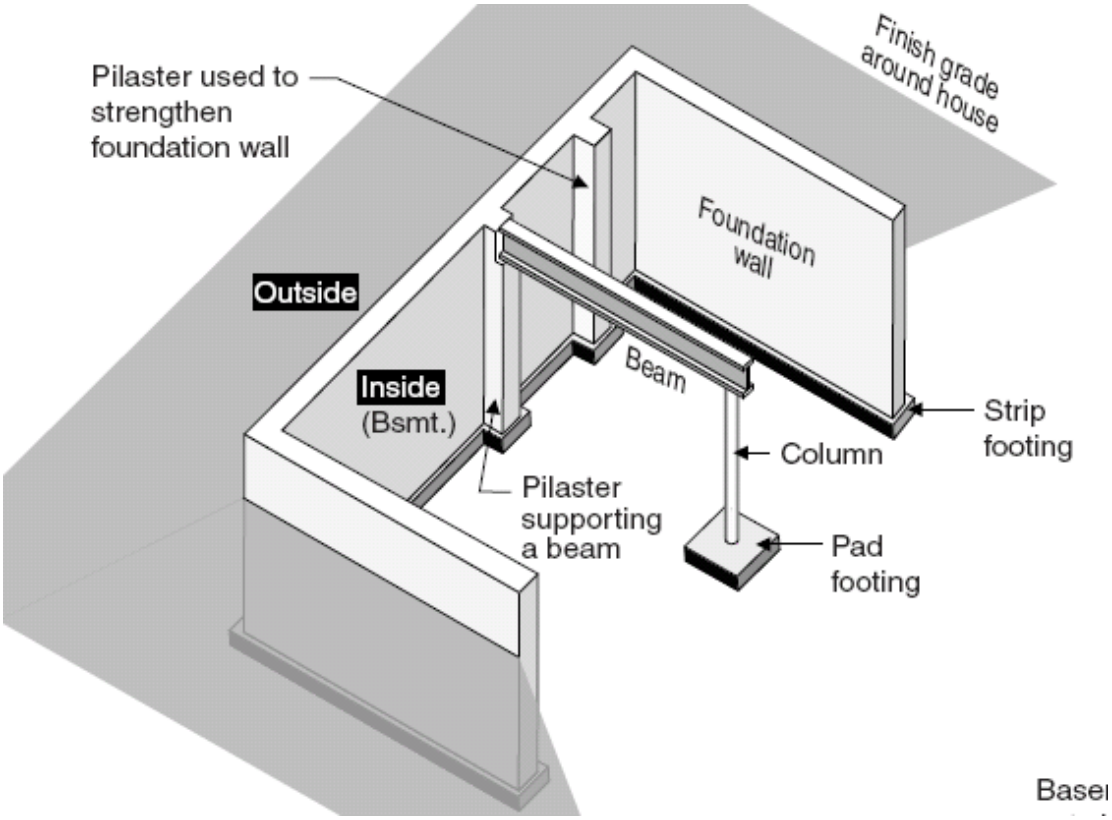


Figure 2.1 isolated column footing and masonry wall footing of low rise building

2.4 Seismic Design of Foundation

The foundations are arguably the most important part of the building. So buildings must be founded in good ground. A building on poor foundations cannot perform well in an earthquake. In order to achieve reliable earthquake resistance the form of construction should be decided from consideration of the following factors: (1) Simplicity and symmetry, (2) Length of plan, (3) Shape in elevation, (4) Uniformity and continuity, (5) Stiffness, (6) Failure mode, and (7) Foundation conditions. Earthquake repeatedly demonstrated that the simplest structures have the greatest chance of survival. There are three main reasons for this. First, our ability to understand the overall behavior of a simple structure is markedly greater than it is for a complex one, e.g. torsional effects are particularly hard to predict on an irregular structure. Second, our ability to understand simple structural details is considerably greater than it is for complicated ones. Third, simple structures are likely to be more buildable than complex ones. Lack of symmetry produces torsional effects which are

sometimes difficult to assess and can be very destructive. Length and shape of plan should be regular for better seismic resistant.

The basic rule regarding the earthquake resistance of substructure is that integral action during earthquakes should be obtained. This requires adequate consideration of the dynamic response characteristics of the superstructure and of the subsoil. If a good seismic-resistant form has been chosen for the superstructure then at least the plan form of the substructure is likely to be sound, i.e., (1) Vertical loading will be symmetrical, (2) overturning effects will not be too large, and (3) The structure will not be too long in plan. As with non-seismic design, the nature of the subsoil will determine the minimum depth of foundations. In earthquake areas this will involve consideration of the following factors:
(a) Transmission of horizontal base shears from the structure to the soil, (b) Provision for earthquake overturning moments (e.g. tension piles), (c) Differential settlements, (d) Liquefaction of the subsoil, and (e) The effect of embedment on seismic response.

The effects of depth of embedment are not fully understood at present, but some allowance for this effect can be made in soil structure interaction analyses, or when determining at what level to apply the earthquake loading input for the superstructure analysis. Isolated column footing require more detailed considerations in order to ensure satisfactory integral action which deals with so many of the structural requirements implied in (1) to (3) and (a) to (e) above. Integral action should provide sufficient reserves of strength to deal with some of the differential ground movements. Also where a change of soil type occurs under a structure particular care may be necessary to ensure integral substructure action. Integral action in isolated footing can be achieved by providing strap or by combining two or more footing, but these options are costly. In continuous wall footing better integral action obtained by continuous RC base, masonry wall, short column and grade beam, also this footing cost is comparatively less than the isolated column footing.

Before completing the design of the foundations it is assumed that the dynamic characteristics of the subsoil have been determined and a suitable form for the substructure should also have been chosen. It then remains to design the foundations

for appropriate seismic forces which arise (1) directly from the deformation of the adjacent soil and (2) as a result of the earthquake forces acting in the superstructure. While our ability to estimate the seismic forces from (2) above is now quite advanced, there remains a great deal of uncertainty about the magnitude and effect of the forces induced directly by the ground. This is true despite the increasing attempts to elucidate the soil-structure interaction problem by sophisticated analytical and experimental techniques.

In current design practice it is often found convenient to consider two separate stress systems: (1) the seismic vertical stresses (e.g. due to overturning moments) and (2) the seismic horizontal stresses (e. g. due to base shear on the structure). Overturning moments are not usually a problem for buildings as a whole, unless it is very slender, but can be difficult for individual footings such as column pads or shear wall strip footings. The foundations should, of course, be proportioned so as to keep the maximum bearing pressures due to overturning moments and gravity loads within the allowable seismic value for the soil concerned. Unfortunately there is little agreement on what constitutes safe seismic bearing pressures on sedimentary soils. Most earthquake codes do not discuss the effect of soil type on bearing pressures. It appears that most soils are capable of sustaining higher short-term loads than long-term loads, with the exception of some sensitive clay which loose strength under dynamic loading.

2.4.1 Analysis for seismic vertical stresses

Foundation should be strong enough to prevent bearing capacity failure. A bearing capacity failure is defined as a foundation failure that occurs when the shear stresses in the soil exceed the shear strength of the soil. For both the static and seismic cases, bearing capacity failures of foundations can be grouped into three categories:

i) General shear: A general shear failure involves total rupture of the underlying soil. There is a continuous shear failure of the soil from below the footing to the ground surface (Fig 2.2 a). When the load is plotted versus settlement of the footing, there is a distinct load at which the foundation fails, and this is designated as Q_{ult}. The value of Q_{ult} divided by the width and length of the footing is

considered to be the ultimate bearing capacity of the footing. The ultimate bearing capacity has been defined as the bearing stress that causes a sudden catastrophic failure of the foundation. General shear failure ruptures and pushes up the soil on both sides of the footing. A general shear failure occur soils that are in dense or hard state.

ii) Punching shear: A punching shear failure does not develop the distinct shear surfaces associated with a general shear failure. For punching shear, the soil outside the loaded area remains relatively uninvolved, and there is minimal movement of soil on both sides of the footing. The process of deformation of the footing involves compression of soil directly below the footing as well as the vertical shearing of soil around the footing perimeter (Fig. 2.2 b). A punching shear failure occurs for soils that are in a loose or soft state.

iii) Local shear: Local shear failure involves rupture of the soil only immediately below the footing. There is soil bulging on both sides of the footing, but bulging is not as significant as in general shear (Fig. 2.2 c). Local shear failure can be considered as a transitional phase between general shear and punching shear. A local shear failure occurs for soils that are in a medium or firm state.

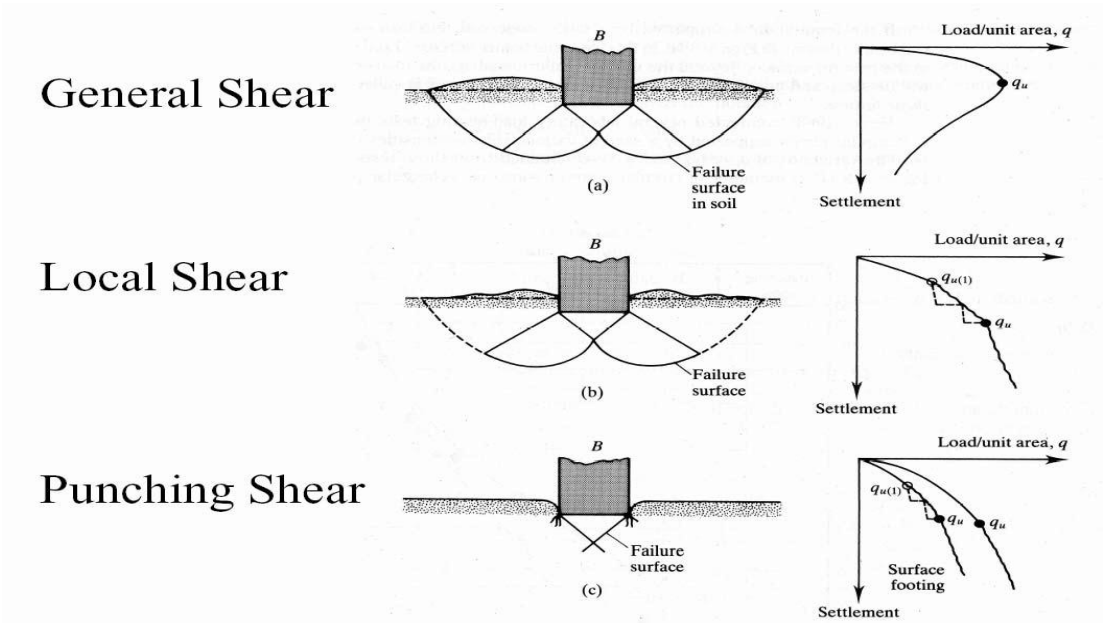


Figure 2.2: Bearing capacity failures; (a) General shear failure, (b) Local shear failure, and (c) Punching shear failure.

Compared to the number of structure damaged by earthquake- induced settlement, there are fewer structures that have faced earthquake-induced bearing capacity failure. This is because of the following factors (Baidya, 2007):

i) Settlement governs: The foundation design is based on several requirements. Two of the mains considerations are that (1) settlement due to the building load must not exceed tolerable values and (2) there must be an adequate factor of safety against a bearing capacity failure. In most cases, settlement governs and the foundation bearing pressures recommended by the geotechnical engineer are based on limiting the amount of expected settlement due to the static or seismic cases. In other cases where settlement is too high, the building is often constructed with a deep foundation, which also reduces the possibility of a bearing capacity failure.

ii) Extensive studies: There have been extensive studies of both static and seismic baring capacity failures, which have led to the development of bearing capacity equations that are routinely used in practice to determine the ultimate bearing capacity of the foundation.

iii) Factor of safety: To determine the allowable bearing pressure q_{all}, the ultimate bearing capacity Q_{ult}, is divided by a factor of safety. The normal factor of safety used for static bearing capacity analyses is 3. According to Baidya (2007), for the evaluation of the bearing capacity for seismic analysis, the factor of safety is often in the range of 5 to 10. These are high factor of safety compared to other factors of safety, such as only 1.5 for slope stability analyses.

iv) Minimum footing size: Building codes often require minimum footing sizes and embedment depths. Larger footing sizes will lower the bearing pressures on the soil and reduce the potential for static or seismic bearing capacity failures.

v) Allowable bearing pressures: In addition, building codes often have maximum allowable bearing pressures for different soil and rock conditions. Table 6.3.5 in BNBC-1993 shows some bearing capacity of different soil for lightly loaded structures.

vi) Footing dimensions: Usually the structural engineer will determine the size of the footing by dividing the maximum footing load by the allowable bearing pressure. Typically the structural engineer uses values of dead, live, and seismic loads that also contain factors of safety. Thus the load that is used to proportion the footing

also contains a factor of safety, which is in addition to the factor of safety that was used to determine the allowable bearing pressure.

The above bearing capacity failures during earthquakes indicate that usually the following three factors (separately or in combination) are the cause of the failure:

i) Soil Shear strength: Common problems include an overestimation of the shear strength of the underlying soil. Another common situation leading to a bearing capacity failure is the loss of shear strength during the earthquake, because of the liquefaction of the soil or the loss of shear strength for sensitive clays.
ii) Structural load: Another common problem is that the structural load at the time of the bearing capacity failure was greater than that assumed during the design phase. This can often occur when the earthquake causes rocking of the structure, and the resulting structural overturning moments produce significant cyclic vertical thrusts on the foundation elements and underlying soil.
iii) Change in site conditions: An altered site can produce a bearing capacity failure. For example, if the groundwater table rises, then the potential for liquefaction is increased. Another example is the construction of an adjacent excavation, which could result in a reduction in support and bearing capacity failure.

The most common cause of a seismic bearing capacity failure is liquefaction of the underlying soil. Settlement analysis and bearing capacity analysis is required to account for the loss of shear strength of the underlying liquefied soil layer. Ground movements or settlement may occur due to liquefaction, consolidation or altering soil properties due to earthquake. Localized failure due to punching when the footing is overloaded could also be the cause for seismic bearing capacity failure. Possibility of foundation settlement is less in continuous wall footing rather than conventionally designed isolated column footing due to a continuous composite body action obtained in wall footing which has larger stiffness in differential movement. But isolated column footing under a column can be settled more due to no tie action provided at the base level which leads to increase differential movement. So it can be said that, wall footing is more seismic resistant than isolated column footing in aspect of bearing capacity failure.

2.4.2 Analysis for seismic Horizontal stresses

The horizontal interaction stresses between the soil and the foundation are arguably more problematic than the vertical stresses, as comparatively little is known about allowable seismic passive pressures and the effect of seismic active pressure in different foundation situations (Baidya, 2007**)**. Indeed it is customary to assume even more arbitrary distributions for horizontal stress between foundations and soil than for vertical stress. The main problems of foundation design as presently understood occur in transferring the base shear of the structure to the ground, and in maintaining structural integrity of the foundation during differential soil deformations. Shallow foundations are often of a form that is highly vulnerable to damage from differential horizontal and vertical ground movements during earthquakes. It is therefore good practice even in quite low structure, especially those founded on soft soils, to provide ties between column pads (Baidya, 2007**)**. This tie action effectively resists the base shear. So tie action obtained in continuous wall footing by RCC base, masonry wall, short column and grade beam. On the other hand, normally isolated column footing has no tie action.

The horizontal seismic shear force at the base of the structure must be transferred through the sub-structure to the soil. With shallow foundations it is normal to assume that most of the resistance to lateral loads is provided by friction between the soil and the base of the members resisting horizontal load. Other footings and slabs in contact with ground may also assumed to provide shear resistance if they are suitably connected to the main resisting elements. The total available resistance to lateral movement of the structure may be taken to be equal to the product of the dead load carried by the elements considered and the coefficient of sliding friction between the soil and the substructure. However, it may be possible to resist some or all of these horizontal forces by passive action of the soil, particularly for light buildings (Baidya, 2007**)**.

As per BNBC-1993, equivalent static force method is applicable for low rise building with regular structure. Seismic base shear is mainly dependent on fundamental period of structure, structural system modification factor and seismic dead load. Fundamental period and structural system modification factor are smaller in confined method rather

than conventional bare frame method. Dead load variation is smaller. So obviously base shear calculated is greater in confined method. This base shear is distributed in the superstructure at the joints only. As a result, structure in confined method is analyzed and designed with maximum lateral loads and base shear rather than conventional method. So foundation of confined method (continuous wall footing) was designed by taking maximum base shear, maximum overturning moments to make the foundation better seismic resistant than conventionally designed isolated footing. Also shear resistance of confined infill wall in continuous wall foundation resist a considerable amount of base shear to make the foundation safer in earthquake. So benefits of confined infill wall can be utilized to develop an economic, seismic resistant foundation system as well as structural system.

Earthquake induced foundation settlement, liquefaction, soil foundation interaction, earthquake resistant foundation design methodology, foundation type selection for low rise buildings were reported by a number of researchers (Hsieh, 2002; The structural committee of FPA, 2004; Patil, 2004; Pender, Wotkerspoon and Ingham, 2005; Geotechnical Manual of Texas, 2006; Baidya, 2007; Augenti and Parisi, 2008; Willford, Whittaker and Klemencic, 2008; Yi, 2010).

2.5 Definition of Confined Masonry Construction

As early as 1960s, studies have been carried out to study the influence of infill on the moment resisting frames under lateral loads induced by earthquake, wind and the blast. Numerous experimental and analytical investigations have been carried out; nevertheless, a comprehensive conclusion has never been reached due to complex nature of material properties, geometrical configuration, and the high cost of computation. Though the effect of infill is widely recognized, there is no explicit consideration in the present code, thus the design engineer's end up designing buildings based on judgment i.e. neglecting effect of infill in design.

Confined masonry construction system is such that where masonry wall is bonded first then Frame structure is constructed i.e. masonry wall first then RCC frame. Confined masonry construction consist of masonry infill, horizontal and vertical RC

confining elements built on all four sides of a masonry wall panel as shown in figure 2.3. Vertical elements called tie column, resemble columns in RC frame construction except that they tend to be of far smaller cross sectional dimension. Most importantly, these RC members are built after masonry wall has been completed. Horizontal elements called tie beams resemble beam in RC frame construction, but they are not intended to function as conventional beams since confined masonry walls are load bearing.

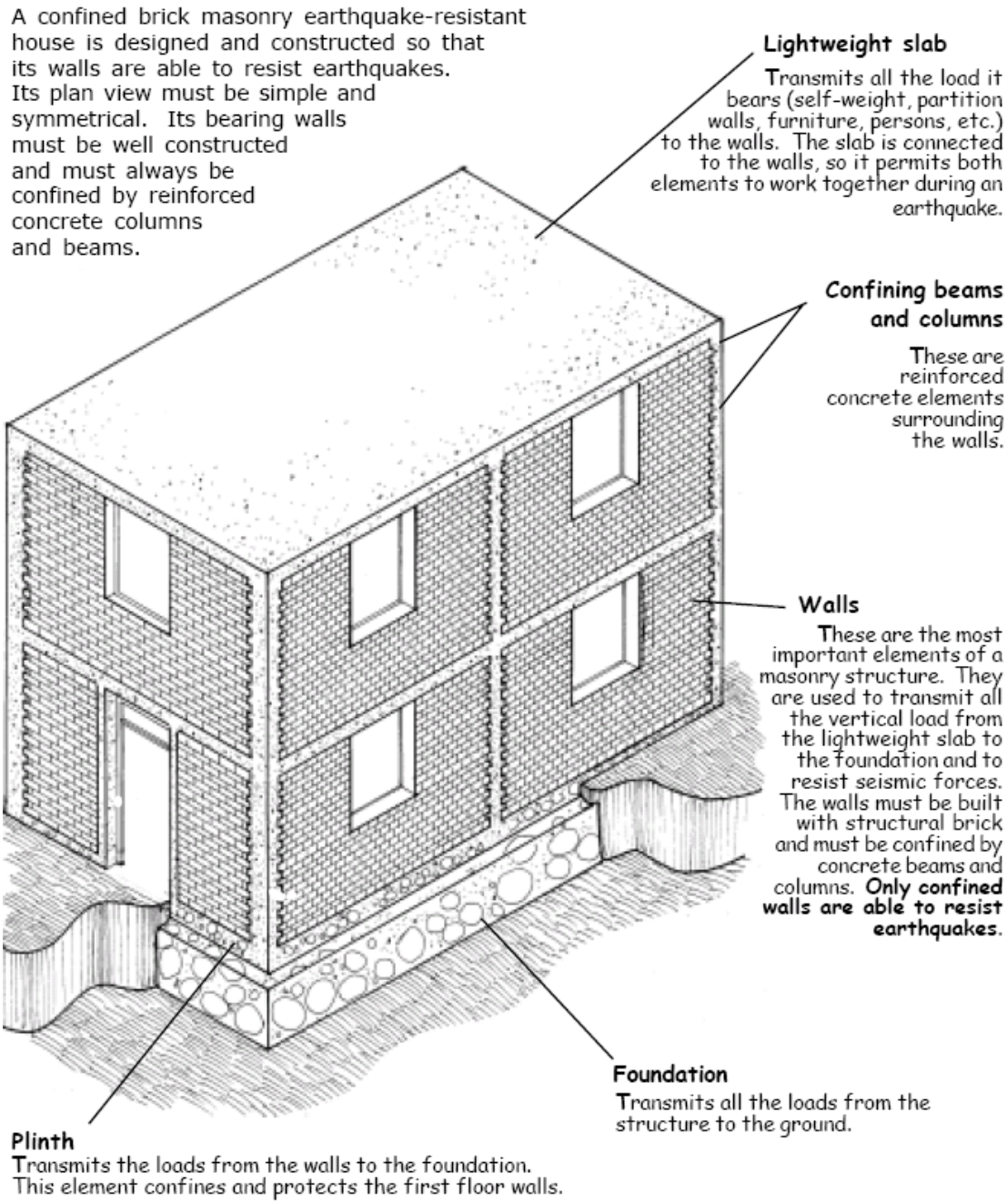


Figure 2.3: Components of Confined Masonry Construction (EERI, 2004)

Masonry wall transmit gravity loads from the slab above down to the foundation along with RC tie beams and columns. The walls acts as a bracing panel which resist horizontal earthquake forces acting in plane. Tie columns and beams prevent brittle seismic response of masonry wall and protect them from complete disintegration even in major earthquake. Floor and roof transmit gravity loads and lateral loads to walls. In an earthquake, floor and roof slab acts as horizontal beams and is called diaphragm. Grade beams/ plinth bands transmit loads from walls to foundation. It also protect ground floor wall from excessive settlements in soft soil condition and the moisture penetration into the building. Foundation transmits all loads to soil. Foundation confined infill wall protect the building from excessive settlement, tilting during seismic ground motion.

RC Frames with masonry infill wall and Confined Masonry: A Comparison

The appearance of finished confined masonry and a RC frame infilled with masonry wall panels may look alike, however these two construction sequence are substantially different. The main difference are related to i) the construction sequence ii) the manner in which these structures resist gravity and lateral loads.

The differences related to construction sequence are as follows:

- In confined masonry construction masonry wall are constructed first, one floor at a time, followed by the cast–in-place RC tie columns. Finally RC tie beams are constructed on top of the walls, simultaneously with the floor/roof slab construction.
- In conventional RC frame construction, the frame is constructed first, followed by the masonry wall construction. So gap found in joint of frame and infill.

It is important to explain, why seismic response of confined masonry buildings is different from conventional RC frames with masonry infill. The main reasons are summarized below:

- Due to smaller cross-sectional dimensions, RC tie columns in confined masonry construction are slender and cannot provide an effective frame action. Beams and columns in RC frame construction are much larger in size and they have significantly larger stiffness relative to the infill.

- Tie-columns are cast against a rough (toothed) surface and thus are integrated into the masonry wall in confined walls in confined masonry construction. On the contrary, infill walls are usually not integrated into RC frame, there is no tooting and there are rarely any dowels.
- Gravity loads in confined masonry construction are mostly supported by the masonry walls; while infill's in RC frames bear mostly self weight. Due to significant frame stiffness, only a small portion of the floor load is transferred to the infills. Also in infill construction it is not uncommon to have gaps between masonry walls and the beams. These gaps allow beams to deflect without transferring the gravity loads to the wall below. RC frames resist both gravity and lateral loads through their relatively large beams, columns, and their connections. Masonry infills are not load-bearing walls.
- When subjected to lateral seismic loads, walls in confined masonry buildings act as a shear walls, similar to RC shear walls. On the other hand, infill wall panel in RC frame building do not act as a shear wall- they act a diagonal struts. The gaps due to relative lack of bond between masonry infill and RC frame drastically minimize the capability of infill walls from resisting the lateral forces in a seismic event.
- In confined masonry construction, confining elements are not designed to act as a moment-resisting frame; as a result, detailing of reinforcement is simple. It is easier to perform an inspection of concrete construction in confined method. Reasons for this include the fact that the reinforcement in confined masonry construction is much simpler, lower strength concrete can often be used. Due to lower consumption of steel and concrete, construction of a confined masonry building is expected to be more economical compared to an conventional RC frame building with masonry infill (particularly in developing countries where labor is relatively inexpensive).

2.6 Modeling of Infill Frame

Model developments in any structures are crucial to achieve accurate output results. However it is difficult to model the as built structures due to numerous constrains with as it is difficult to incorporate all physical parameter associated with the behavior

of infilled frame structure. Even all physical parameters, such as contact coefficient between infill and frame, separation and slipping between the two components and the orthotropic of the material properties are considered, there is no guarantee that the real structure behave similar to model as they also the structural behavior could also depend on the quality of materials and construction techniques.

However, there are several analytical methods to predict the behavior, strength and stiffness of infilled frames. These analytical methods can be grouped into two categories (i) Macro Model (ii) Micro Model.

2.6.1 Macro Model

Macro models try to generate the force deformation characteristics of infill. These methods usually idealize the panel by an equivalent diagonal beam or compression strut. Although these methods require less computational effort, they are usually valid only for the tests for which the derivation is made. The drawbacks of it are to the lack of capability to consider the opening precisely found in the infill walls. Modeling of infill wall can be done by the methods (i) Crossed-diagonal Strut (ii) Two-diagonal Strut (iii) Single diagonal Strut approach. But for simplicity in analytical modeling, single diagonal Strut is the most common. Figure 2.4 c shows the macro model.

The concept of diagonal strut was initially introduced by Polyakov (1960) while investigating a three storey infilled frame. The cracks along the length of panel gave an insight of the strut behavior of an infill panel. The report stated that stress from peripheral frame members to the infill was transferred only through the compression corners of the frame –infill interface.

Holmes (1961) proposed the width of the equivalent strut to be one third of the diagonal length from his experimental study on a single story single bay infilled structure under in-plane loads. Smith(1962) conducted a study on a specimen of had steel frame and concrete mortar as infill and observed a compression region within the infill panel which made the frame stiff and thus concept of diagonal strut was evolved.

Mainstone (1971) proposed the strut area, A_e in in^2 was:

$A_e = w_e t$ (2.1)

$w_e = 0.175(\lambda h)^{-0.4} w'$

$$\lambda = \sqrt[4]{\frac{E_i t \sin(2\theta)}{4E_f I_c h'}} \quad (2.2)$$

Where,

E_i= the modulus of elasticity of the infill material, ksi

E_f= the modulus of elasticity of the frame material. ksi

I_c=the moment of inertia of column, in^4

t=the thickness of infill wall, in

h=the centre line height of frame

h'=the height of infill

w'=the diagonal length of infill panel

θ=the slope of infill diagonal to the horizontal

Paulay and Priestley (1992) proposed the width of equivalent strut was

$w_e = 0.25 d_m$ (2.3)

where,

w_e=width of strut

d_m=the length of the infill diagonal strut.

FEMA 356 (2000) also suggest compression strut model for infill modeling irrespective of opening effects.

Although the diagonal strut model have gained popularity in modeling and analysis of infilled structures, it is only suitable to study the global structural response. This approach does not count the effect of opening in infill walls. However the micro model i.e. finite element techniques is the most preferred method for most of the researchers as it allows to understand both local and global responses.

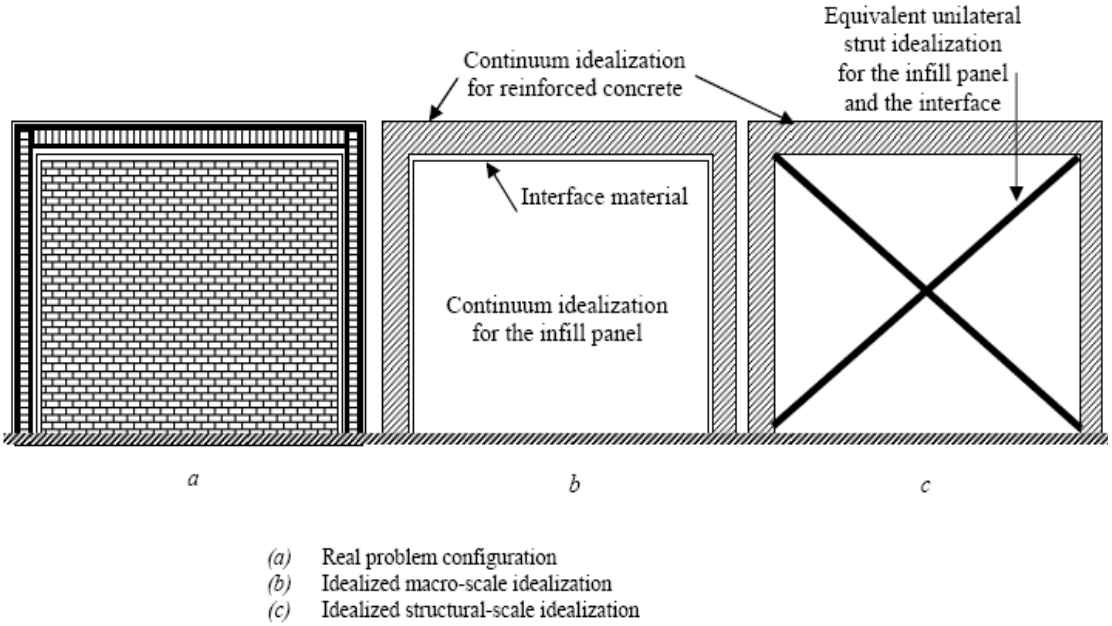


Figure 2.4: Methodologies for development of infill panel Model (Taher and Afefy, 2008)

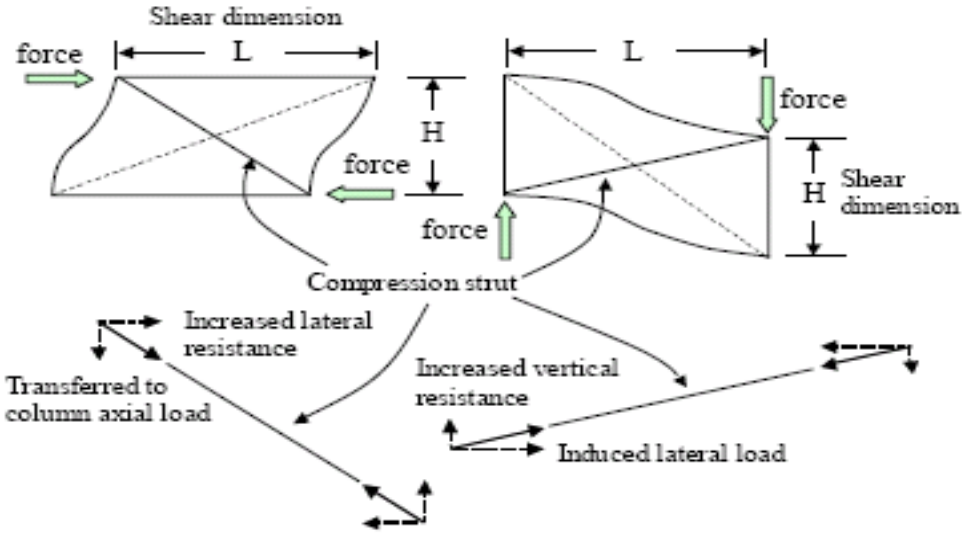


Figure 2.5: Role of compression strut under horizontal and vertical loadings

2.6.2 Micro Model

A Finite Element (FE) method is a process of discretezing the structural components into a smaller size, maintaining the constitutive laws of materials, boundary conditions, in order to improve the accuracy of the results. This method employs principles of mechanics of solids to model the frame and the infill behavior. This method suggests continuum idealization of infill panel among the RC frame members

and interface materials is assumed to exist in between infill wall and RC elements. In this method opening in infill panel can be easily model to count actual behavior of infill. Infill panel and frame members are divided into finite element mesh. Large computational efforts are required to obtain meaningful results and taking comparatively longer time. But this is the more accurate method to determine the behavior of infill. This method has the advantage of modeling the real structure and using 3D modeling. Micro model method improved by experimental findings will be used more extensively in the future.

In this method bricks were assumed as linear elastic brittle material, mortar can be considered as an elastic material susceptible to damage, understood as degradation of stiffness and strength. Interface can be modeled as non-integral continuum material with no tensile capacity and brittle behavior in compression using appropriate interface elements. In this study, gap element was used as interface elements in ETABS model between infill and reinforced concrete. Reinforced concrete can be modeled as a unilateral nonlinear isotropic hardening elasto-plastic material, where the behavior in compression and tension is different.

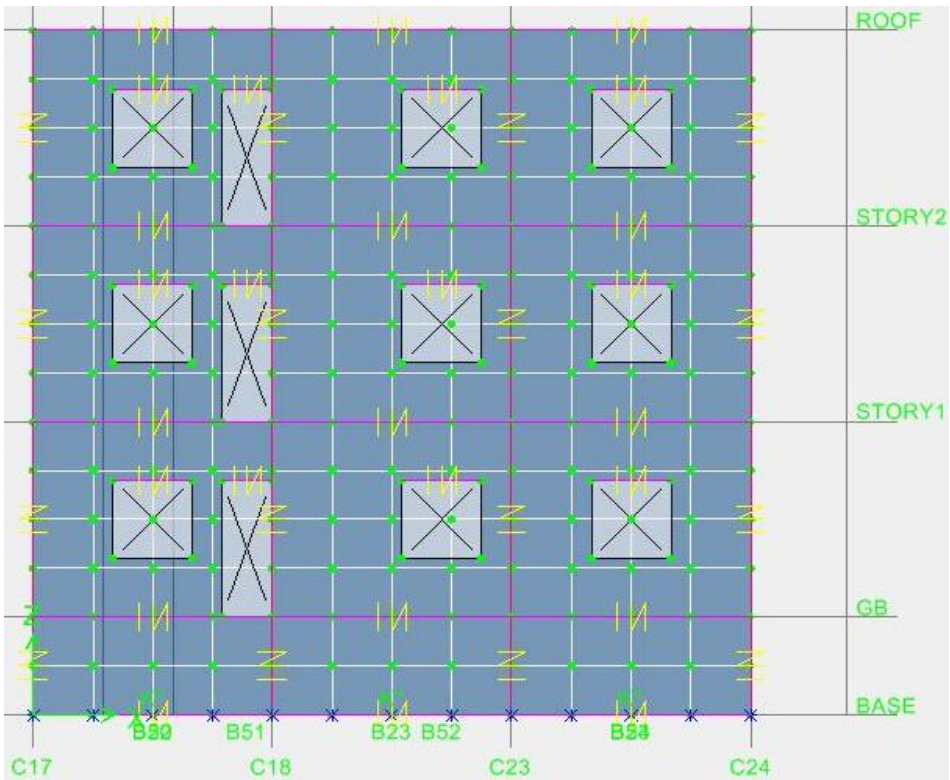


Figure 2.6: Finite Element Model

Figure 2.4 b shows the modeling elements of this method. To make a FE model, 3D software is generally required.

The finite element model with and without a perfect contact between the infill wall and RC frame was studied by Combescure and Pegon et al (1995) on a single bay single storey structure. It was reported, under unilateral contact condition (frictionless) the forces between the infill and frame are transferred through a compression corners at the ends of diagonal strut. However there is no transfer of shear from infill to frame. When a perfect contact condition considered at the interface, shear force transfer between two.

Haddad (1991) studied the application of a Finite Element method to assess the effects of cracking and separation between the frame and infill of an infilled structure. It has been found that bending and deflection decreases with the increase in infill frame relative stiffness. Bending moment further increased with the crack depth. The moment at an un-cracked section increased when the crack size on other end was increasing. The magnitude and location of principal compressive and tensile stresses were affected by crack size, contact length and infill frame relative stiffness. However the author recommended the good use of materials and construction techniques to reduce damage due to separation and cracking.

Similar research on the infilled structures using FE techniques were carried out by Morbinducci, (2003); Saneinejad, (1990); Seah, (1998); Lourenco, (1996); Singh, (1998). However most of them had investigated on a single storey models under in-plane static loads.

Although the diagonal strut model have gained popularity in modeling and analysis of infilled structures, it is only suitable for the study of global structural response. Also this method does not include effects of opening conventionally exist in building in the analytical modeling. However the Finite element (FE) techniques are the most preferred method for most of the researchers as it allows understanding both local and global responses. The micro models are the most accurate approach, because they can take into consideration the contact interface conditions, various material laws for the

frame and infill (elastic, elasto-plastic, fracturing, etc.) and several other modeling parameters, while their results tend to the “exact” response of the physical structure as the discretization mesh becomes finer. Also FE model is resemblance to real structure, so it can be said that FE techniques are more accurate than diagonal strut approach of infilled frame analysis.

2.7 Analysis Type

There are different types of analyses to treat the seismic force on structure. Most code specify both static and dynamic analysis, with the choice based on a number of considerations such as importance of building, its height, the soil and the seismic hazards at the location based on past events. The static analysis is the indirect method of considering the effect of ground motion on the structure and it normally incorporates some of dynamic features of the problem, such as fundamental period of the building, the soil effect and earthquake hazard. Dynamic analysis like time history analysis is the direct method in which a selected earthquake records in the form of acceleration-time history is used as the input. Time history analysis can be used for both linear and nonlinear analysis. There is also a pseudo dynamic analysis method called response spectrum analysis in which the relevant periods of a building are used to obtain the acceleration to be applied to the structure.

2.7.1 Static Analysis

As per BNBC-1993, static analysis can be perform the structure regular or irregular in seismic zone-I and standard occupancy structure in zone –II except the soil profile S_4 or T>0.7 seconds. Also this method can be applicable to irregular structures not more than 20 meter height or regular structure up to 75 meter height. This method of seismic analysis involved the distribution of total base shear throughout the height of the structure. The base shear is found based on the seismic co-efficient which is based on the seismic hazard exposure for a particular location and total weight of the structure. Although this method is static, there is an incorporation of dynamic properties of the structure in terms of fundamental period and response reduction factor. However this method is limited to a regular type structure whose maximum

response is governed by the first mode of vibration. Fundamental period of infilled structure must be calculated by suing the formula for masonry shear wall stated in section 2.5.6.2(a).

If the infill walls are considered in the modeling and analysis, most of the structured lower than ten stories in height would give maximum response at first mode (Dorji, 2009). This is due to an additional stiffness contributed by the infill which eventually makes the structure stiffer and rigid.

2.7.2 Response Spectrum Analysis

This method uses the peak modal responses obtained from dynamic analysis on a single degree of freedom (SDOF) system. The peak acceleration is found for different periods for the model and plot of spectral acceleration versus period gives the curve which is called response spectrum curve. This curve is generally very rough but the codes recommend the smoothened curve. The values for low range of period are kept constant while it is varied for high period models. It is not necessary to use code specified spectrum if the site specific spectrum is available.

This technique is extended to multi-degree of freedom system (MDOF) by performing linear superimposition of modes shapes using the modal combination techniques such as SRSS (square roots of the sum of squares) and CQC (complete quadratic combination). The disadvantage of SRSS is its incapability of considering the modes that are very close unlike the CQC. The result from this analysis gives only the peak structural responses at desired damping values.

2.7.3 Time History Analysis

The time history analysis involves a time-step by step integration of dynamic equilibrium equation. The general equation for a dynamic response of a multi degree of freedom system subjected to ground motion is given by

$$[M]\{\ddot{x}(t)\}+[C]\{\dot{x}(t)\}+[K]\{x(t)\}=\{F(t)\} \qquad 2.4$$

Where,

[M] = Mass matrix,
[C] = Damping matrix,
[K]= Stiffness matrix,
$\ddot{x}(t)$=Acceleration
$\dot{x}(t)$=Velocity
x(t)=Displacement
F(t)=The inertial force due to earthquake.

The solution for this equation can be achieved by performing numerical integration methods such as Newmark Integration method, Wilson-θ method and Runje-kutta forth order method. The structural responses are computed at each step and thus equation 2.4 is solved. The stability criteria of the numerical method are conditional for explicit algorithm but it is unconditionally stable for implicit algorithm. When the algorithm is unstable the higher mode shapes dominate the structural response and it is unacceptable, especially for medium rise building. However when the time step size is too small the computational time lengthen and becomes uneconomical. A time step of the order Tn/10 or smaller would be adequate where Tn is the period of nth mode and all modes up to the nth modes will then participate in the analysis. The chosen time step provided adequate convergence of results.

2.7.4 Viscous damping

It is a force that resists motions at all times. Ideally there are no structures which do not have damping because all structures are in one way or other posses force on account of frictional force in the joints, air resistance and the frictional force within the molecule of the material. Damping is the linear combination of mass matrices and the stiffness matrices. In general 5% damping is considered critical for reinforced concrete structure, 3% for unreinforced masonry structures and 7% for masonry structures (Chopra, 2000). However the damping ratio for confined infilled frame

structure could not be found in any of the literature as well as code. So author suggested viscous damping of confined masonry as 3% and viscous damping of reinforced concrete frame as 5% can be used in the model.

2.8 Influence of Confined masonry infill in Earthquake Resistance

Confined masonry infill in RC frames acts as a diaphragm in vertical plane that imparts significant lateral strength and stiffness to RC frames under lateral loads. Infilled frames also tend to be substantially stronger, but less deformable, than otherwise conventional bare frames. In symmetric buildings with vertically continuous confined infilled frames, the increased stiffness and strength may protect a building from damage associated with excessive lateral drift or inadequate strength. Because of its higher stiffness, infill panels may attract significantly greater forces that may lead to premature failure of infill, and possibly of the whole structure. Therefore, it is essential for designers to consider the effects of infills in the design of RC buildings.

Earthquake loads can be resisted by:

- Moment-resisting frames
- Braced frames
- Shear walls

Frames infilled with masonry wall (recommended for low rise only).

Among all RC frames infilled with masonry wall (confined construction) is the most economic and effective system of combination to resist earthquake loads.

2.8.1 Earthquake Resistant Mechanism of Confined masonry wall and infill wall

Significant experimental and analytical research effort has been expended till date in understanding the behavior of masonry infilled RC frames. Seismic behavior of a confined masonry wall panel can be explained by composite (monolithic) action of a masonry wall and adjacent RC confining elements. Infill wall is the brick masonry wall constructed after frame construction.

When subjected to lateral seismic loads, walls in confined masonry buildings react at the same time with the RC confining elements like tie column and beams and act as a unique body when deflect as like shear walls, similar to RC shear walls as shown in figure 2.7 (b). On the other hand, infill wall panels in RC frame buildings do not act as shear walls - they act as diagonal struts as shown in figure 2.7 (a).

When lateral load applied to infilled frame, a single diagonal compression strut forms as shown in figure 2.7 (a) and infill walls separated from frame along the other diagonal i.e. at the other two corners due to the gap of frame and infill. As thus the lateral load transfer mechanism changes from the predominant frame action into truss action (Murty and Jain, 2000). The gaps, that forms due to the construction process (frame first and then infill wall) i.e. due to a relative lack of bond between the masonry infill and the RC frame, drastically minimize the capability of infill walls from resisting the lateral forces in a seismic event. The frame columns now experience increased axial load but reduced bending moments and shear forces. The infill wall is so much weak in out-of-plane seismic loading. This diagonal strut has crushed after very short run of seismic loads due to the separation or gap of frame and infill and then total lateral load has applied to the frame. So the predominant frame action again has achieved in infilled RC frame (referred as conventional bare frame in this paper) after failure of infills under lateral loading. Plastic hinge develop in the beams and columns near joint as shown in figure 2.8 (a). That’s why greater size and more rebar used in design of infilled frame members as it carry all gravity and lateral loads.

Confined masonry wall in RC frame plays a vital role in resisting the lateral seismic loads on buildings. When lateral load applied to the confined building, diagonal cracking develop along the diagonals of the infill panel as shown in figure 2.8 (b). This crack propagates into the tie columns at higher load level as shown in figure 2.9. This indicates that compression diagonal strut form along the two diagonal of the infill panel. This behavior is experimentally confirmed by Tomazevic and Klemenc (1997); Marjani and Ersoy (2002); Zabala, Bustos, Masanet, and Santalucia (2004);

Zabala, Honma, Gibu and Gallardo (2004); Hashemi and Mosalam (2006); Pujol, Climent, Rodriguez and Pardo, (2008).

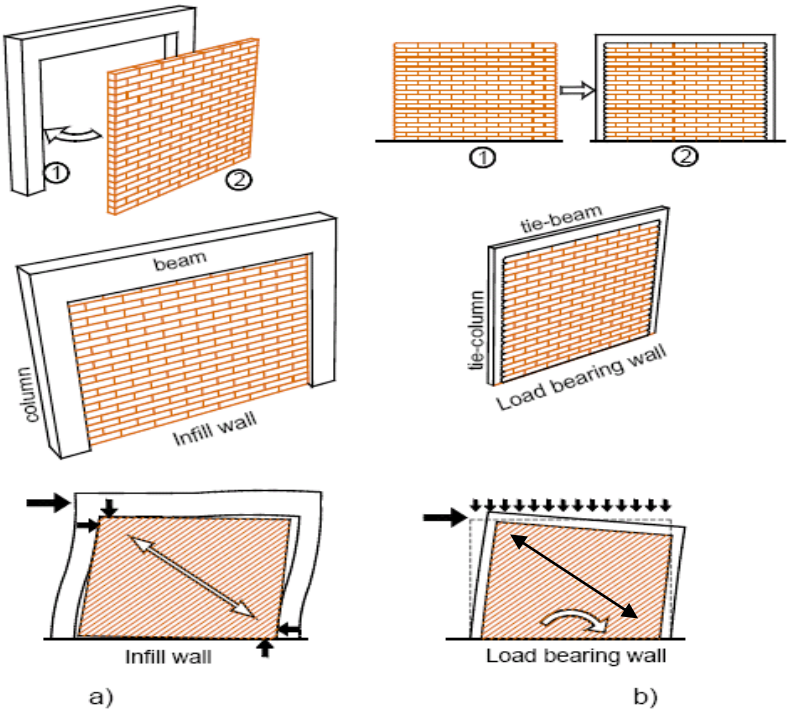
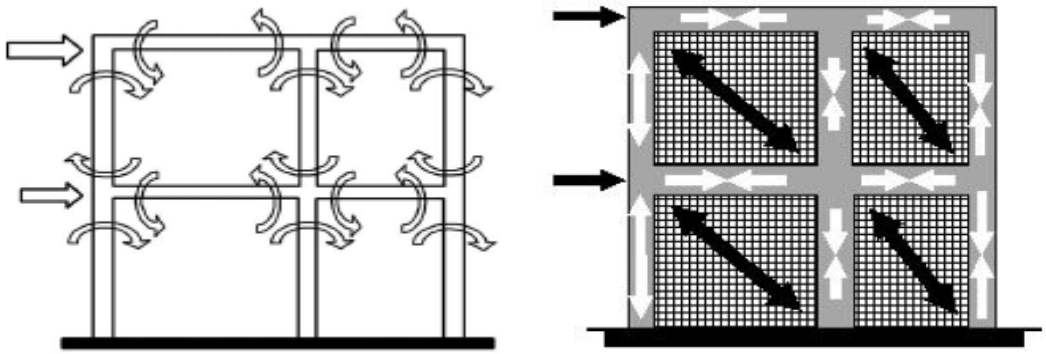


Figure 2.7: A comparison of (a) RC frame with masonry infill's (b) Confined Masonry construction; construction sequence (top); size of confining elements (middle), and the seismic response (bottom). (Brzev, 2010 and www.confinedmasonry.org)

As thus the lateral load transfer mechanism changes from the predominant frame action into truss action (Murty and Jain, 2000) as shown in figure 2.8. This truss action is responsible to reduce bending moments and increase axial load to the confining elements. Due to the construction process (infill first and then frame) i.e. due to a relative good bond between the masonry infill and the RC frame, the confined masonry construction drastically increase the capability of infill walls from resisting the lateral forces in a seismic event and act as an integrated body to reduce the differential settlement of foundation, deflection of the structure, increase the lateral stiffness, ductility, shear capacity, energy dissipation capacity. Initially a confined masonry panel resists the effects of lateral earthquake loads by itself while the confining elements (tie column) do not play a significant role. However, once cracking takes place, the wall pushes the tie-columns sideways. At that stage vertical

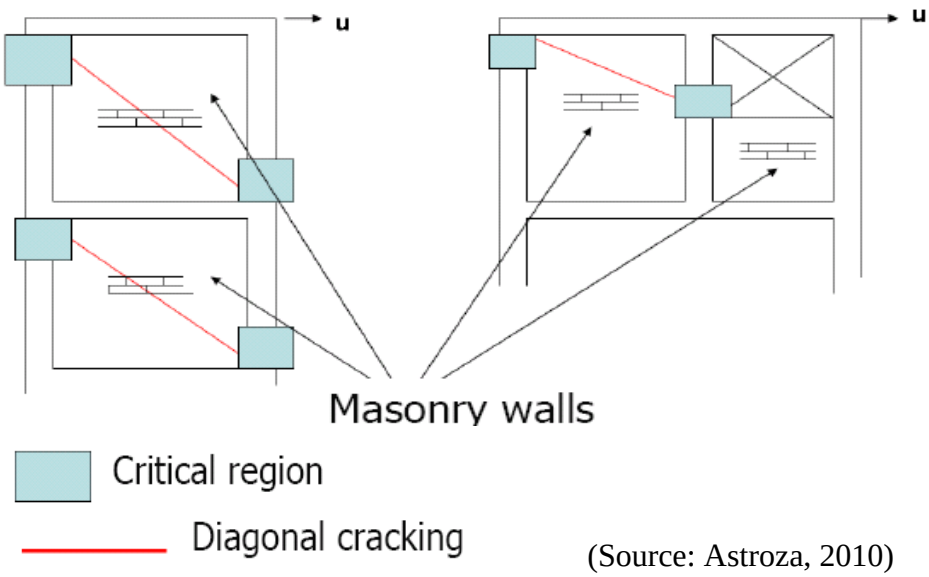
reinforcement in tie columns engaged in resisting tension and compression stresses (Tomazevic and Klemenc, 1997).



a) Predominant Frame action b) Predominant Truss action

Figure 2.8: Change in lateral load transfer mechanism due to confined infill wall

So confined masonry walls act as diagonal struts subjected to compression, while the RC confining members act in tension and/or compression, depending on the direction of lateral earthquake forces. This model is appropriate before the cracking in the walls takes place. Damage in the tie columns at the ultimate load level is concentrated at the top and bottom of the panel can be referred as critical region as shown in figure 2.9. These locations, characterized by extensive crushing of concrete and yielding of steel reinforcement, are called plastic hinges. In confined masonry construction, tie beams and tie columns resist axial loads. Shear failure can lead to severe damage in the masonry wall and the top and bottom of the tie columns.



(Source: Astroza, 2010)

Figure 2.9: Mechanism of seismic response in Confined Masonry Building

Shear capacity of a confined masonry wall panel (3) can be determined as the sum of contributions of the masonry wall (1) and the adjacent RC tie-columns (2), as shown in Figure 2.10. Note that the shear capacity of tie-columns can be reached only after the masonry has been severely cracked and its shear capacity has significantly decreased. As a result, it is recommended to consider only a partial contribution of tie-columns to the shear capacity of a confined masonry panel. It can be seen from the diagram in Figure 2.10 that the stiffness and strength of a confined masonry panel drop following the onset of diagonal cracking in the wall (point 1). However, the load-resisting capacity of the panel is still maintained until the critical regions of the confining elements experience significant cracking (point 2). This shows that a significant lateral deformation and ductility can be attained before the failure of a properly designed and constructed confined masonry panel (point 3).

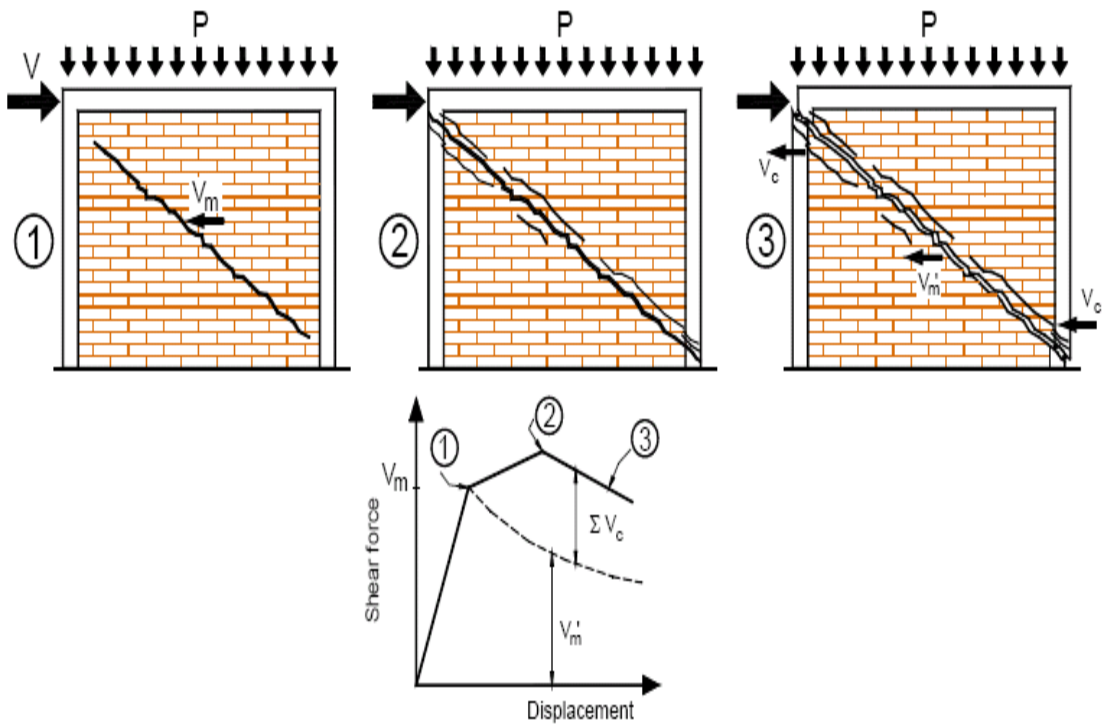


Figure 2.10: Mechanism of shear resistance for a confined masonry wall panel 1) diagonal cracking in the masonry wall; 2) diagonal cracks have propagated from the wall into the tie-columns, and 3) shear failure of the RC tie-columns and the confined masonry wall panel. (www.confinedmasonry.org)

Tie column also resist the major portion of gravity load when masonry walls suffer severe damage. The failure of tie column usually takes place when cracks propagate from the masonry wall into the tie column and shear it off. Subsequently the vertical stability of the entire wall is compromised. Thus tie column have a critical role in resisting the gravity load in damaged confined masonry buildings and ensuring their vertical stability (Alcocer, et al, 2004).

Flexural failure caused in confined masonry by in-plane lateral loads is characterized by horizontal cracking in the mortar bed joints on the tension side of the wall (Yoshimura et al. 2004) as shown in figure 2.18**.** Separation of tie columns from the wall due to flexural effect was observed in some cases (when toothed wall-to-column connection was absent). In most cases, confined masonry panels demonstrate a shear-dominant seismic response. Longitudinal reinforcement in the RC tie-columns provides an adequate flexural resistance, thus the flexural failure mechanism does not govern.

Confined masonry panels are subjected to the effects of axial gravity load (due to self-weight and tributary floor/roof loads). Figure 2.11 (a) illustrates a confined masonry panel which resists the combined effect of axial load P and bending moment M. The capacity of the composite confined masonry panel section under the combined effect of axial load and bending moment can be determined by treating the confined masonry panel similar to a RC shear wall acting in unison/companion with the adjacent columns. The strain diagram shows that a portion of the panel is in tension, while the remaining portion is in compression as shown in figure 2.11 (b). It is assumed that the masonry and concrete are not able to resist tension; hence tensile stresses are resisted by the longitudinal reinforcement in tie-columns. The compression stresses are resisted by concrete, masonry, and longitudinal reinforcement in tie-columns as shown in figure 2.11 (c). The flexural capacity of the panel section is determined from the sum of moments created by various internal forces around point O (Centroid of the section).

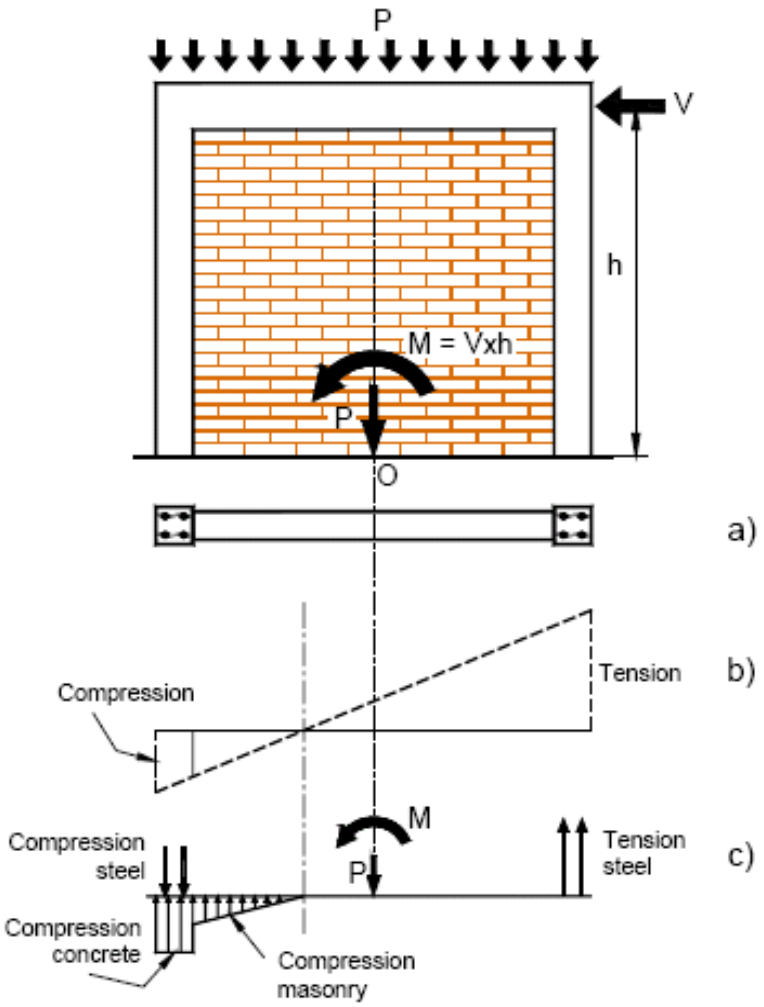


Figure 2.11: A confined masonry wall panel subjected to the combined axial load and bending a) panel elevation and a typical cross-section; b) strain distribution, and c) internal force distribution. (www.confinedmasonry.org)

2.8.2 Seismic Design Principle

The fundamental principle of seismic design is to minimize the loss of lives and property in the event of disastrous earthquake. This is being ensured by providing enough strength, stiffness and ductility in the structures and the ductility of the structural member could be achieved by proper reinforcing details at the appropriate location. However infill walls are treated as non-structural component and its significant contribution on the structure is ignored conventionally. Consequently, this could result to unacceptable performance of structure during earthquakes as the presence of infill alters the behavior of frame structures.

To control the sudden failure of entire structure, a concept of “Strong column weak beam” has been introduced in the most seismic codes worldwide, wherein beams are made to undergo damage before the columns during ultimate loading condition. The displacement based design concept has gained popularity due to better understanding of nonlinear behavior of reinforced concrete materials under the earthquake loads. Thus the failure of beams is preferred rather than the column failure. This is achieved by introducing the strength factor on the column moments at the joints.

Accordingly, the sum of column moment at a joint should be 6/5 times the sum of beam moment as given by Pauley (1992) in equation 2.5.

$$\Sigma M_c = 6/5 \Sigma M_b \qquad 2.5$$

Where,

M_c=the column moment at the joint,

M_b=the beams moment at the joint

Both the beams and columns are provided with ductile detailing so that the structure fails in desired manner as shown in figure 2.12. Allowing the beam elements to dissipate energy prior to columns to control the abrupt failure more realistic in case of the bare frame system but there is no much research information available on infilled frame designed to this concept.

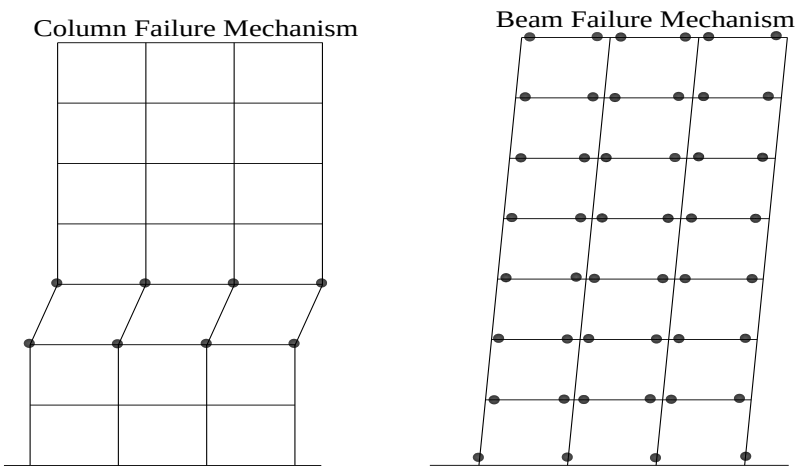


Figure 2.12: Failure Mechanism in Reinforced Concrete Frames.

Parducci (1980) have reported that strong infill with weak frame undergoes premature failure of columns and the report was based on the test performed on a single storey infilled frame. However, there is no enough repot on the infilled frame designed to "strong column weak beam concept" (CEB, 1996; Pauley, 1992). However this concept may not be applicable to infilled RC structure as the presence of infill alter the global behavior of the structural system, besides increasing the possibility of structural failure due to soft storey mechanism during strong earthquake.

The presence of infill would alter the stiffness of the members (Pauley, 1992), thus some members get over stressed for which they are not designed, are liable to suffer damages. Therefore, there is a need to consider the effects of infill on frame structure.

Infills possess large lateral stiffness and hence draw a significant share of the lateral force. When infills are strong, strength contributed by the infills may be comparable to the strength of the bare frame itself. According to Murty and Jain, (2000), the mode of failure of an infilled building depends on the relative strengths of frame and infill (Table 1). And, its ductility depends on the (a) infill properties, (b) relative strengths of frame and infill, (c) ductile detailing of the frame when plastic hinging in the frame controls the failure, (d) reinforcement in the infill when cracking in infills controls the failure, and (e) distribution of infills in plan and elevation of the building.

Table 2.1: Modes of failure of masonry infilled RC frames

	Weak Infill	Strong Infill
Weak Frame	-	• Diagonal cracks in infills • Plastic hinges in columns
Frame with Weak joints and strong Members	• Corner crushing of infills • Cracks in beam-column joints	• Diagonal cracks in infill • Cracks in beam-column joints
Strong Frame	• Horizontal sliding in infills	-

According to Doudoumis (2007), when $K_i/K_c \leq 10$ it was flexible or weak infill and $K_i/K_c=100$ can be termed as stiff infill where K_i and K_c is infill stiffness and columns stiffness respectively.

When infills are non-uniformly placed in plan or in elevation of the building, a hybrid structural load transfer mechanism with both frame action and truss action may develop (Murty and Jain, 2000). In such structures, there is a large concentration of ductility demand in a few members of the structure. For instance, the soft storey effect (According to BNBC-1993, a soft storey is the one in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above), the short column effect (when infills are raised only up to a partial height of the columns), and plan-torsional effect (when infills are un-symmetrically located in plan), cause excessive ductility demand on frame columns and significantly alter the collapse mechanism. These behaviors are generally found in RC bare frame with masonry infill wall.

Earthquake-induced lateral forces in multi-story confined masonry buildings, peak at the ground floor level and may cause significant shear cracking. In a confined masonry building the stiffness is initially equal at all floor levels. Under severe earthquake ground shaking, the collapse of a confined masonry building may take place at the first story level, as shown in Figure 2.13 due to cracking is concentrated at the ground floor roof level under high seismic loads which may cause extensive masonry cracking and a resulting decrease in the lateral stiffness. This behavior was confirmed by experimental studies (Ruiz and Alcocer, 1998; Alcocer et al., 2004, 2004a). Note that this mechanism is somewhat different from the soft-story collapse mechanism which is found in RC frames with masonry infill walls, but both building fails at first storey level. To control this failure, key factors influencing the seismic resistance have to consider in seismic design like wall density, mortar strength, tie column, wall toothing, horizontal wall reinforcement and opening and ductile detailing have to follow as per any stabilized code. Also RC confining elements must be designed to prevent crack propagation from the walls into critical regions of RC confining elements.

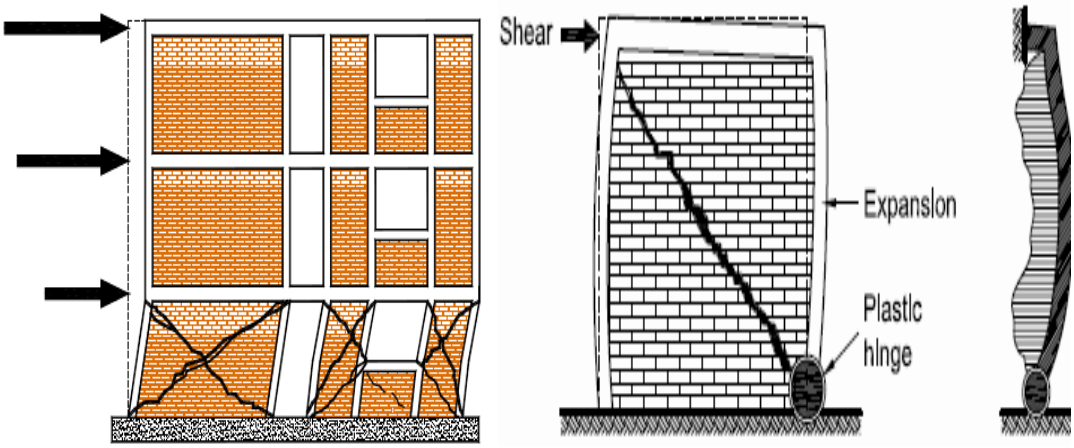


Figure 2.13: Collapse mechanism for multi-story Confined masonry buildings (Alcocer et al. 2004)

According to ATC-40 (1996), there are five categories of structural performance levels such as immediate occupancy (sp-1), damage control (sp-2), life safety (sp-3), limited safety (sp-4), and the structural stability (sp-5). Similarly there are four categories of non-structural performance levels such as operational (np-1), immediate occupancy (np-2), life safety (np-3), and reduced hazard (np-4). These performance levels are for the post earthquake assessment based on the physical observation. Since there are many types of infill materials, performance level would vary and thus important to study the suitability of an infill material for intended purpose.

2.8.3 Key factors influencing Seismic Resistance of Confined Masonry Structures

2.8.3.1 Wall Density:

Wall density is believed to be one of the key parameters influencing the seismic performance of confined masonry buildings. Evidence from past earthquakes show that confined masonry buildings with adequate wall density were able to resist the effects of major earthquakes without collapse. It can be determined as the transverse area of walls in each principal direction divided by the total floor area of the building. In Mexico, a simplified procedure was developed to determine the required wall density for buildings in which the wall seismic resistance is governed by shear effects (Meli, 1994). After the 1985 Mexico earthquake, the Mexico City Building Code

imposed more stringent wall density requirements for masonry buildings. These requirements were based on the fact that, due to the reality of Mexican construction practice, it was not possible to prescribe the use of higher strength materials or the provision of reinforcement as an alternative approach for increasing earthquake resistance. Therefore, after the 1985 earthquake, the code required a 40% increase in the design seismic resistance; this resulted in a significant increase in wall density requirements in confined masonry construction. As an example, for a five-storey building in Mexico City, it is now required to provide wall density of around 6% in each direction, whereas in the areas of highest seismic risk, this value is close to 10%.

As per Eurocode8 minimum wall density, P_A% requirement for confined masonry buildings is shown in Table 2.2.

Table 2.2: Minimum Wall Density of Confined Masonry Building (Eurocode8)

Site acceleration a_g.S		< 0,07 k ·g	< 0,10 k ·g	< 0,15 k ·g	< 0,20 k ·g
Type of construction	Number of stories	Minimum sum of cross-sections areas of horizontal shear walls in each direction, as percentage of the total floor area per storey ($p_{A,min}$)			
Confined masonry	2	2,0%	2,5%	3,0%	3,5%
	3	2,0%	3,0%	4,0%	n/a
	4	4,0%	5,0%	n/a	n/a
	5	6,0%	n/a	n/a	n/a

* n/a means "not acceptable".

$k = 1 + (l_{av} - 2)/4 \leq 2$ where l_{av} is the average length, expressed in m, of the shear walls considered (k = 1 for other cases)

A comprehensive study done in Chile after the 1985 earthquake showed that the extent of damage in masonry buildings was related to the wall density in the following way: buildings with a wall density of less than 0.5% suffered severe damage, while the buildings with wall density of 1.15% suffered only light damage. The survey performed in Chile showed that over 50% of surveyed confined masonry buildings have a wall density of over 1.15%.

2.8.3.2 Masonry Units and Mortar

The past tests have shown that the lateral load resistance of confined masonry walls strongly depends on the strength of the masonry units and the mortar used. The walls built using low-strength bricks or un-grouted hollow block units had the lowest

strength while the ones built using grouted or solid units had the largest strength. However, the use of grouted and solid units results in an increase both in wall mass and seismic loads. Also, the weaker the mortar the lower the masonry strength (due to the unit-mortar interaction, the masonry strength is always lower than the unit strength). Test results have also shown that there is no significant difference in wall strength between unreinforced and confined masonry wall specimens with the same geometry and material properties (Alcocer and Klingner, 1994).

Masonry units should have sufficient robustness in order to avoid local brittle failure. As per Eurocode8, except in cases of low seismicity, the normalized compressive strength of masonry units, derived in accordance with EN 772-1, should be not less than the minimum values as follows:

- Normal to the bed face $f_{b,\,min}$ = 5 N/mm^2
- Parallel to the bed face in the plane of the wall $f_{bh,\,min}$=2 N/mm^2

Minimum compressive strength of mortar for confined masonry should be 5 N/mm^2.

2.8.3.3 Tie-Columns

Tie-columns significantly influence the ductility and stability of cracked confined masonry walls. Experimental studies have shown that vertical strains in the confined masonry walls decrease at an increased damage level, thereby indicating that a major portion of the gravity load is resisted by tie-columns. Note that the effect of tie-columns on increasing lateral resistance of confined masonry structures has only recently been recognized (Alcocer, 2006). The provision of closely spaced transverse reinforcement (ties) at the top and bottom ends of tie-columns results in improved wall stability and ductility in the post-cracking stage (Alcocer and Klingner, 1994).

2.8.3.4 Horizontal Wall Reinforcement

In many countries where confined masonry construction is practiced, reinforcement is usually not provided in masonry walls. However, in four-to-five storey construction in Peru there is a tendency to provide horizontal joint reinforcement in the form of one or two wires laid in the mortar bed joints (Casabonne, 1994), as shown in Figure 2.14.

The Mexican Code NTC-M 2004 prescribes that the horizontal reinforcement, when provided, be placed continuously along the wall length. Horizontal rebar should be anchored into the tie-columns; the anchorage should be provided with 90° hooks at the far end of the tie-column (see Figure 2.14). The hooks should be embedded in the concrete within the tie-column (note that the tie-column reinforcement was omitted from the figure). The bar diameter should be larger than 3.5 mm and less than ¾ the joint thickness.

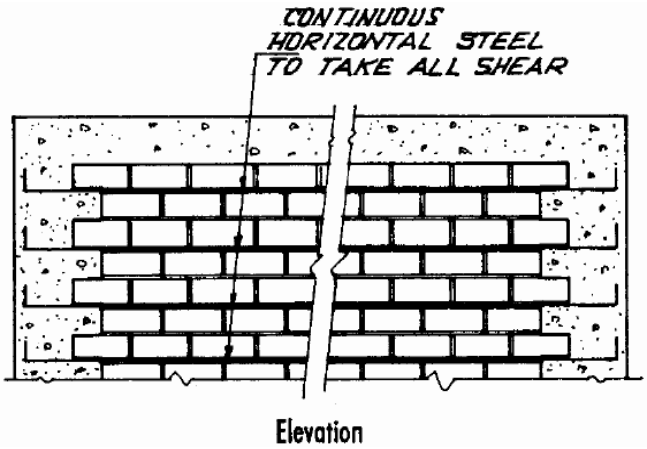


Figure 2.14: Horizontal reinforcement in Confined masonry walls (Casabonne, 1994)

Research studies have shown that horizontal reinforcement has a beneficial effect on wall ductility. Specimens with horizontal reinforcement showed a more uniform distribution of inclined shear cracks than the unreinforced specimens. Recent Mexican codes (starting from NTC-M 2002 and followed by NTC-M 2004) have recognized the contribution of horizontal reinforcement to the overall shear strength of confined masonry walls. A special efficiency factor has been introduced to account for the effect of the horizontal reinforcement ratio; its values are based on experimental studies, which have revealed that the horizontal reinforcement is less efficient in heavily reinforced walls. It should be noted that cold-drawn steel wires are used as horizontal reinforcement in Mexico; these wires are made of steel without a defined yield plateau, where strain hardening develops at very small strains (0.002 to 0.0025) (Alcocer, 2006; Alcocer et al. 2003). An effective way to avoid the fragile behavior associated with the soft storey like effect is to provide horizontal reinforcement in masonry walls in order to enhance their shear resistance (Schultz, 1994). The type of steel used for horizontal reinforcement influences its effectiveness in enhancing

masonry shear resistance. Early experimental studies used horizontal reinforcement made of high carbon steel that exhibited elasto-plastic behavior. These studies showed that the provision of horizontal reinforcement does not result in a significant strength increase (Alcocer and Klingner, 1994; Aguilar et al. 1996). Also, the specimens with intermediate tie-beams placed at the wall mid height, did not show satisfactory behavior, since the wall panel above the intermediate tie-beam caused the failure in tie-columns. In some countries, like Costa Rica (Casabonne, 1994) and El Salvador (EERI, 2001a), intermediate tie-beams are used in confined masonry construction practice.

2.8.3.5 Openings

An experimental research study showed that, when the opening area is less than approximately 10% of the total wall area, the wall lateral load resistance is not significantly reduced as compared to a solid wall (Yanez et al. 2004). Opening significantly decreases the lateral strength of confined masonry (Liauw, 1977; Liauw, 1992). The walls with larger openings develop diagonal cracks (same as solid walls), except that the cracks are formed in the piers between the openings; thus, diagonal struts form in the piers, as shown in Figure 2.15. The study recommends estimating the lateral strength of walls with window and door openings based on the net transverse wall area (equal to the wall thickness times the wall length reduced by the sum of window lengths). Note that, in this study, the vertical reinforcement bars were provided around the door openings. Most building codes prescribed the maximum permitted opening size beyond which the tie-columns need to be provided. Eurocode8 specify provision of tie column for opening greater than 1.5 m^2.

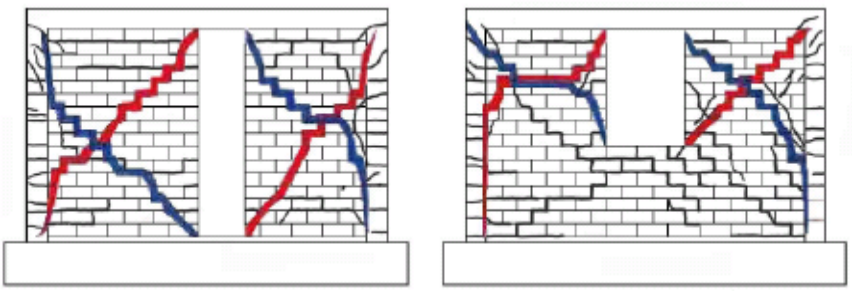


Figure 2.15: Failure modes in the confined masonry walls with openings (Yanez et al., 2004)

2.8.3.6 Toothing at the Wall-to-tie-column Interface

Good bonding between a masonry wall and adjacent RC tie-columns is important for satisfactory earthquake performance, and for delaying undesirable cracking and separation at the wall-to-tie column interface. Bonding is an essential feature of confined masonry construction and it can be achieved by toothing at the wall-to-tie-column interface. Toothing plays a vital role in out-of-plane seismic resistance of confined masonry construction. Toothed edges should be left on each side of the wall at the interface with the tie-columns. Toothing length should be equal to one-quarter of the masonry unit length, but not less than 5 cm. Horizontal reinforcement anchored into RC tie-columns, also known as dowels, can be used as an alternative to toothing.

2.8.4 Seismic behavior of RC frames with Masonry infill

Numerous experimental and numerical investigations carried out in past have proven that the presence of infill improves significantly the lateral strength of an infilled frame system. The parameters in increasing strength are strength of infill materials, strength of confining elements, relative stiffness of infill to frame ration, presence of opening, reinforcement of infill panel, strength of mortar and masonry unit, lack of initial fit between infill and the frame etc.

The presence of gap created between infill and frame during conventional construction decreases the lateral load resistance capacity (Mainstone, 1971; Schmidt, 1989). It is also known that the presence of infill in the frame increases the lateral stiffness of the system by four to twenty times to that of bare frame system (CEB, 1996). Lateral stiffness of building can be increased by introducing infill panel if the structure has problem with drift (Anil, 2006).

There has been much work conducted into the seismic behavior of infilled frame buildings. The complex nature of the interaction between concrete frames and masonry infill wall panels is reflected perhaps in the large number of experimental studies conducted on this topic. For example, Bertero and Brokken (1983) summarized work in which the effects of masonry and lightweight concrete infills on

RC moment resisting frame buildings were studied experimentally and analytically. The experimental investigations consisted of quasi-static cyclic and monotonic load tests of 1/3-scale models of the lower 3.5 stories of an 11 storey, 3-bay RC frame infilled in the outer two bays. The RC frame was designed for high rotational ductility and resistance to degradation under reversed cyclic shear loads. For reasons of economy, ease of construction, favorable mechanical properties and efficiency of different types of masonry infill, it was concluded that the most promising panel configuration consisted of solid brick laid in mortar reinforced with two mats of welded wire fabric, one bonded to each size of the wall in a layer of cement mortar.

Around the same time, Zarnic and Tomazevic (1984) summarized the results of their experimental and analytical investigations into the seismic behavior of masonry infilled RC frames. It was found that the infill began to crack at approximately 0.2% lateral drift and the system exhibited satisfactory behavior up to 2% drift. Small amounts of horizontal reinforcement of the infill were found to have little effect. Subsequently, experimental tests on seismic retrofit methods suitable for RC frames with masonry infill were performed (Zarnic et al, 1986a). The repair methods consisted of epoxy grouting of cracks in the concrete frame and masonry infill elements plus strengthening of the masonry infill by reinforced concrete jacketing of the infill panels.

Valluvan et al. (1994) conducted tests to study the shear transfer across frame-wall joints in RC frame buildings. The tests helped clarify (1) the benefits of compression across the joint, (2) the role of reinforcing bars as dowel shear connectors across the joint, (3) the influence of strength of concrete in the existing frame, and (4) the poor performance of grouted frame-wall joints.

Manos et al. (1995) reported on the results of experimental and analytical investigations into the seismic behavior of RC frames with masonry infill. Bare RC frames reached their maximum strength at drifts of 1%. The infill reached its maximum at 0.3% drift and was able to maintain it until about 2% drift. Based on this work, it was concluded that 3D panel elements could be used to model infill effects

but that realistic material properties are required in order to obtain accurate models of prototype structures.

The ability of masonry infill walls to increase the strength of concrete frames was further highlighted in an experimental study of the seismic behavior of a 1/3-scale concrete frame (1-bay, 4-storey) with infill walls by Liauw and Kwan (1992). Two models were tested. In the first model, the frame was infilled with reinforced concrete shear walls. Brick masonry was used for the infill in the 2nd model without any connectors to the surrounding frame. Both structures had the same static strength. This was even more dramatically demonstrated in a full-scale PSD (pseudo-dynamic testing) test conducted on a 4-storey RC frame (Negro and Verzeletti, 1996). The frame was first tested as a “bare frame” with a nominal acceleration 50% larger than that used for design. The structure responded as a weak-beam strong-column mechanism. The fundamental frequency was half after the tests. The maximum base shear force was 140 kN and the maximum drift was about 2%. The frame was then tested with all bays in the frame infilled with unreinforced hollow block masonry. The same loading histories were used for all tests. The ground-storey infill was severely damaged by the 1.50 times design earthquake test. The maximum drift during this test was 1.1% and the maximum base shear was 2000 kN (approximately 50% stronger than the bare frame).

The results of tests conducted on 12 separate 1-bays, 1-storey RC frames with masonry infill were reported by Mehrabi et al. (1996). The experimental results indicate that the infill can improve the seismic response. Specimens with strong frames and strong infill exhibited much better behavior. The least ductile specimen performed satisfactorily up to 2% drift. For the bare frame, V_{max} occurred at 3.1% drift and still retained 80%V_{max} at 6.8% drift. In contrast, the infilled frames reached V_{max} at 0.5% drift and 80%V_{max} at 1.5% drift.

Mosalam, White and Ayala (1998) used pseudo-dynamic testing (PSD) to study the response of a 2- storey, 2-bay steel gravity load designed steel frame with unreinforced concrete block masonry walls. They showed that it could be successfully modeled analytically using “equivalent” struts with gap elements to model “lack of

fit" between the frame and infill. Hence, the wall did not act until between ± 0.1% and ± 0.2% drift since the hysteretic storey shear versus drift loops all had bi-linear stiffening shapes. The infill walls were badly cracked by drifts of about 0.5% in the PSD tests.

Schneider et al. (1998) conducted experiments to investigate the in-plane seismic behavior of steel frames with URM (unreinforced masonry) brick infill panels with large window openings. The test results indicate: (1) infill strength and stiffness deterioration seemed to be independent of the pier width and wall thickness; (2) ultimate shear strength of the infills ranged between 0.82 and 1.08 MPa at drifts of between 0.75% and 1.5%; and (3) stiffness deterioration for each infill was almost identical. By 0.2% drift each infill had lost more than 70% of its original effective stiffness and all gone by 2% drift. Finally, narrow piers tended to be more ductile than wide piers, resulting in less observable damage than wide piers. In contrast, two brick thick piers tended to be more ductile than single-brick thick piers.

Govindan et al. (1986) conducted tests on a 7-storey concrete frame with brick infill walls in order to study its strength, ductility and energy absorption characteristics as compared to those for the same frame without infill walls. It was observed that the infill frame strength was double that of the bare frame and that its maximum strength was reached at a lateral drift of 3.7%. The bare frame reached its maximum strength at a drift of 1%.

Shing et al. (1992) and Combescure (1996) have developed finite element models based on both a smeared crack and discrete crack approach to evaluate the shear resistance of masonry walls and masonry infilled RC frames. While such models were shown to be capable of reproducing experimental results with a high degree of accuracy, it was observed that the numerical results were highly dependent on the finite element idealization. Furthermore, the inelastic behavior of an infilled frame is very sensitive to the shear strength of the mortar. Shing et al. (2009) found that the maximum load in the infilled specimen occurs at 0.25% drift.

Dukuze (2000) investigated the failures modes of infilled structure on a single storey specimen with and without opening. In general, three types of failure were observed under in-plane loads such as sliding of bed joint, tensile cracking of infill and local crushing of compression corners at the loaded corners. The specimen with opening at the center of panel had suffered shear cracks at the point of contact and severe damages on the lintel beams. It was reported that only piers (infill between opening and column) of specimen exhibited diagonal cracking. However, when the aspect ratio (H/L) was increased, the crack pattern spread throughout the panel and the column fails in shear and bending. The failure of fully infilled specimen was dominate with diagonal cracking along with shear slip along mortar joint. Failure occurred at the loaded corners in most cases, the specimen which had strong column; failure occurred mostly near the beam in the loaded corner and conversely failure concentrated near the loaded region of column when there beam is stronger than column.

Doudoumis (2007) studied the importance of contact condition between the infill and frame members on a single story finite element model. It was reported that the contact condition, friction coefficient, size of mesh, relative stiffness of beam to column, relative size of infill wall have significant influence on the response of infilled frame, while the effect of orthotropy of infill material was reported to be insignificant. It means the infill can be treated as homogenized material. When the mesh density was made finer the stress pattern within the infill was also improved, with maximum value of stresses at the compressive corners. The existence of friction coefficient at the interface was reported to increase the lateral stiffness of the system. However the friction coefficient depend on the quality of materials and the workmanship (CEB, 1996), which is difficult to define accurately.

Eshghi and Pourazin (2009) performed experimental and analytical study to determine the in-plane behavior of confined with and without opening. He carried a pushover analysis over a finite element model of single bay single storey confined building. The Rankin-Hill yield criterion was used with low orthotropic factor to model the masonry panel, the Rankine yield criterion to model reinforced concrete bond-beams and tie-columns, the Coulomb friction criterion with tension cutoff mode to model the

interface zone between the masonry panel and reinforced concrete members. It was reported that the maximum lateral strength and the ductility factor of walls with and without opening are different. Also, the severe compressed zones of the masonry panels within the confining elements are found different from what are reported for the masonry panels of infilled frames by other researchers. This study shows that a further investigation is needed for estimating capacity of confined masonry walls with and without opening analytically and experimentally. Also where openings, with medium size are existed, the confining elements should be added around them.

2.9 Failure Modes of infilled frame

The experimental as well as numerical research performed over last few decades showed different failure mechanism of an infilled frame structure. Most of them have used single storey system under in-plane loads. It has been reported that separation takes place between infill and the frames at the early stage of loading all around the interface excepting the two compressive ends. The angular distortion of the infill studied by Polyakov (1960) varied in the value between $\Delta/h=0.03 \times 10^{-3}$ to 0.7×10^{-3} (where Δ is the horizontal displacement and h is the height of the structure) depending on the relative stiffness of infill to frame stiffness and the external loads. The onset of separation may also depend on the quality of workmanship, lack of fit and material quality. However the prediction of separation is not important as it does not considerably affect the rigidity of the infilled frame (CEB, 1996).

Smith (1996) reported that the weak frame cannot transmit the forces to the compressive diagonal of infill and thus suffers local crushing at the ends of compressive diagonal. On the contrary, the strong frame can transmit high forces to the compressed diagonal which set infill to initiate cracking from the central region and propagated towards the compressed diagonal ends (Mainstone, 1971). It was also reported that when the weaker infill is used with stronger frame system, horizontal sliding failure occurs along the bed joints of mortar (Zarnic et al, 1984). On the contrary, when the stronger infill is used with weaker frame, the frame underwent premature failure of columns before the onset of frame failure (Parducci et al, 1980). It means that infilled frame does not reach to its full capacity.

Generally mortar joints are considered to be planes of weakness due to low shear resistance. Cracks can appear in the interface of column and infill, beam and infill and between the infill elements which gives negative impression on performance of the structures (Miranda Dias, 2007). The shearing failures of joints was reported in research carried out by (Abdou, 2006 and Miranda Dias, 2007) occurred along the plane of weakness.

Mehrabi (1994) observed brittle shear failure of column on windward side while investigating the infilled frame structure which had strong infill panel and weak frame. However the increase in the lateral load resistance was found even after the shear failure of column, indicating some kind of ductility due to infill. On the contrary, the formation of hinges in columns and slip in the bed joints were observed in the weak infill frame test specimen. The stronger frame with stronger infill had failed by crushing of infill as the shear failure of columns was prevented due to enough shear reinforcement and bigger column size.

So the failure modes of confined masonry infilled structure during earthquake are:

- In plane shear failure (diagonal cracking) of masonry wall (sliding shear and bending).
- Out-of-plane failure of masonry wall.
- Shear failure of confining elements.
- Bond failure between masonry and concrete elements.
- Instability of interior walls and failure of unconfined walls and frames.
- Buckling of tie columns.
- Overturning (collapse) of masonry gable wall.

Failure mechanisms in confined masonry wall panels depend on the direction of earthquake loading. There are two possible scenarios:

i) Earthquake ground shaking in the direction parallel with the longitudinal wall axis, also known as in-plane seismic loading, or
ii) Earthquake ground shaking perpendicular to the longitudinal wall axis, or out-of-plane seismic loading.

2.9.1 In-Plane Failure Mechanisms

A confined masonry wall subjected to in-plane lateral earthquake loading develops either a shear or flexural failure mechanism (Tomazevic and Klemenc, 1997; Tomazevic, 1999; Yoshimura et al. 2004).

Shear failure mechanism is characterized by distributed diagonal cracking in the wall. The damage is caused either by the bond destruction at the mortar-brick interface (shear-friction mechanism), or tensile cracking in the masonry units. Initially, a masonry wall panel resists the effects of lateral earthquake loads while the RC tie-columns do not play a significant role. However, once cracking takes place, the wall pushes the tie-columns sideways. At that stage, the vertical reinforcement in the tie-columns resists tension and compression stresses (Tomazevic and Klemenc, 1997).
Damage in the tie-columns at the ultimate load level is concentrated at the top and bottom of the panel. Shear failure can lead to severe damage in the masonry wall and at the top and bottom of the tie-columns, as shown in figure 2.16.

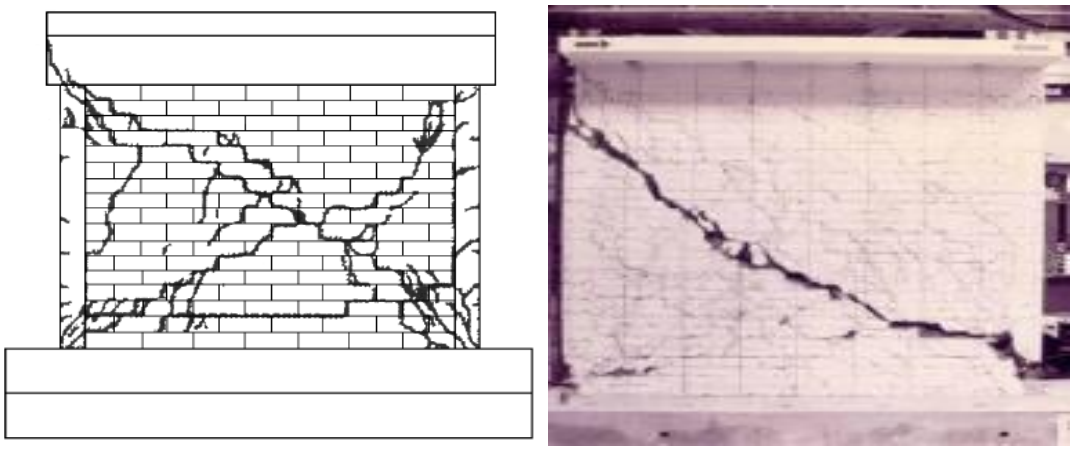


Figure 2.16: Shear failure of confined masonry walls (Yoshimura et al., 2004 – left; Aguilar and Alcocer, 2001 – right).

In-plane shear failure of ground floor confined masonry walls is the most common damage pattern observed in past earthquakes, e.g. the 1999 Tehuacán and the 2003 Tecomán, Mexico earthquakes, the 2001 San Salvador, El Salvador earthquake, and the 2010 Maule, Chile earthquake. Figure 2.17(a) shows damage at the ground floor level of a three-story building in Cauquenes. The building was constructed in 1993,

before the 1997 edition of Chilean code NCh2123, which contains relevant design restrictions for confined masonry buildings, had been issued. Figure 2.17(b) shows shear failure of a pier in a confined masonry building due to the 2001 El Salvador earthquake (absence of RC tie-columns at openings).

a) b)

Figure 2.17 In-plane shear failure of poorly confined masonry walls: a) the 2010 Maule, Chile earthquake (Brzev et al, 2010) and b) the 2001 El Salvador Earthquake (EERI, 2001)

Flexural failure mechanism due to in-plane lateral loads is characterized by horizontal cracking of the mortar bed joints located on the tension side of the wall, as shown in figure 2.18 (Yoshimura et al. 2004). Separation of the tie-columns from the wall was observed in some cases when a toothed wall-to-column connection was absent, and there were no connecting ties between the tie-column and the wall. Extensive horizontal cracking in tie-columns and shear cracking in the walls can be observed in figure 2.18. Flexural mechanism is not as critical as shear mechanism since it does not lead to brittle failure, although crushing and disintegration of masonry in the compression toe area of the wall may take place.

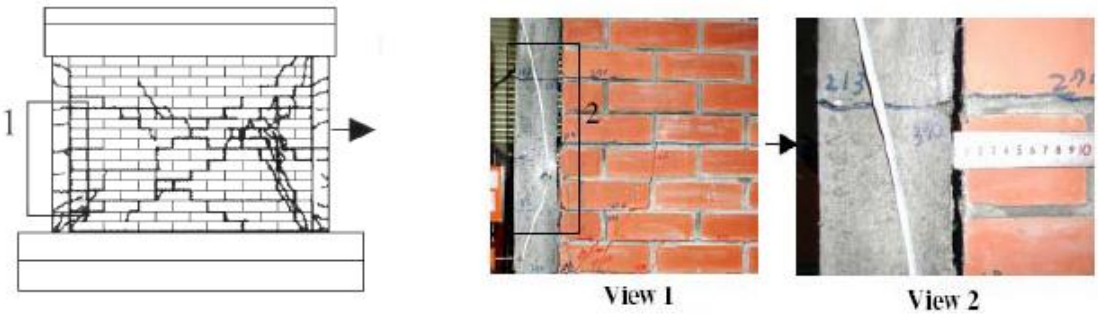


Figure 2.18: Flexural failure of confined masonry walls (Yoshimura et al., 2004).

RC tie-columns have a critical role in resisting the gravity loads in damaged confined masonry buildings, and in ensuring their vertical stability (Alcocer, 2006). Due to their high axial stiffness and tension/compression load resistance, tie-columns resist a major portion of gravity load after the walls experience severe damage. The failure of a tie-column usually takes place when cracks propagate from the masonry wall into the tie-column and shear it off. Note that the failure of tie column could take place either due to the flexural failure mechanism or shear failure mechanism observed in the 2010 Maule, Chile earthquake.

2.9.2 Out-of-Plane Failure Mechanisms

Seismic shaking in the direction perpendicular to a masonry wall (also known as out-of-plane seismic loading) causes bending and shear stresses in the wall. This may result in cracking of the wall and possible collapse by overturning (toppling). Due to an increase in spectral accelerations up the building height, the out-of-plane seismic effects are more pronounced at higher floor levels, as shown in figure 2.19(a). In the area affected by the 2010 Maule, Chile earthquake, wall cracking due to out-of-plane seismic effects was observed at the top floor level, as shown in figure 2.19(b) (no damage was observed at lower floors in the same direction). The building had RC floor slabs and timber truss roof system. (Note that this the same building suffered extensive damage in the longitudinal direction at the ground floor level, as shown in figure 2.17 (a).

The extent of damage and a likelihood of wall collapse depend on the type of roof and floor diaphragm (rigid or flexible), and how well the wall is attached to its confining elements (if any). The out-of-plane bending mechanism is critical mainly for buildings with flexible diaphragms, which are not capable of transmitting the lateral forces to the stiffer walls oriented in the direction of seismic action. In some cases, this mechanism can also be critical in buildings with rigid diaphragms due to inertia forces generated by transverse wall vibrations as shown in figure 2.19 (a).

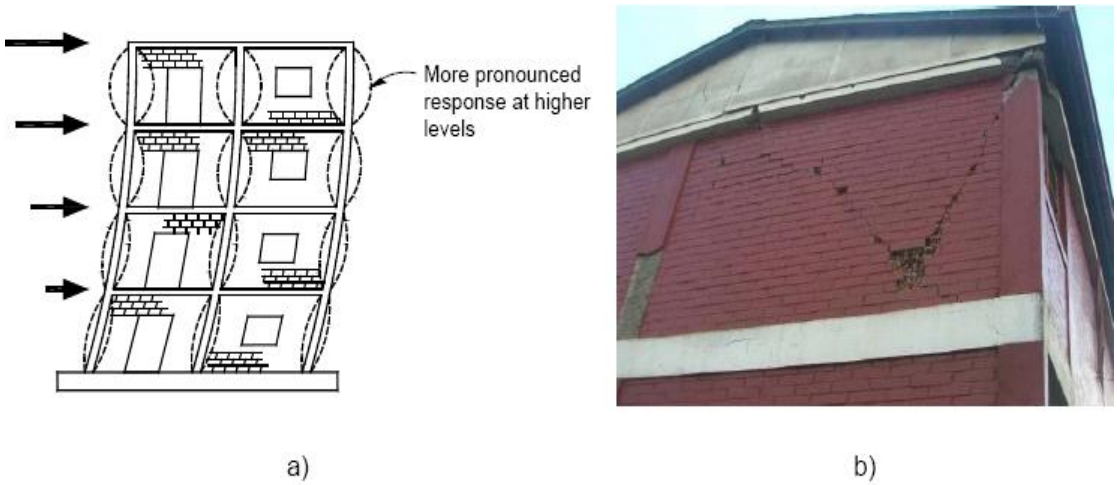


Figure 2.19: Out-of-plane seismic response of confined masonry walls: a) Mechanism of seismic response (Tomazevic, 1999,) and b) observed damage at the top floor level of a building after the 2010 Maule, Chile earthquake (Brzev et al, 2010).

A possible out-of-plane failure mechanism for walls in buildings with rigid diaphragms is similar to that characteristic of a two-way slab supported on all sides and subjected to uniformly distributed loading, as shown in figure 2.20(a). This damage pattern was observed at the second floor level of a three-storey building damaged in the 2010 Maule, Chile earthquake, as shown in figure 2.20(b). However, in companion shake-table tests of a scale-model of the same 4-storey RC building, it was observed that out-of-plane loading was not a problem for the concrete frame with infill (even without mechanical connections) if the gap between the infill and frame was less than 10mm (Negro and Taylor, 1996).

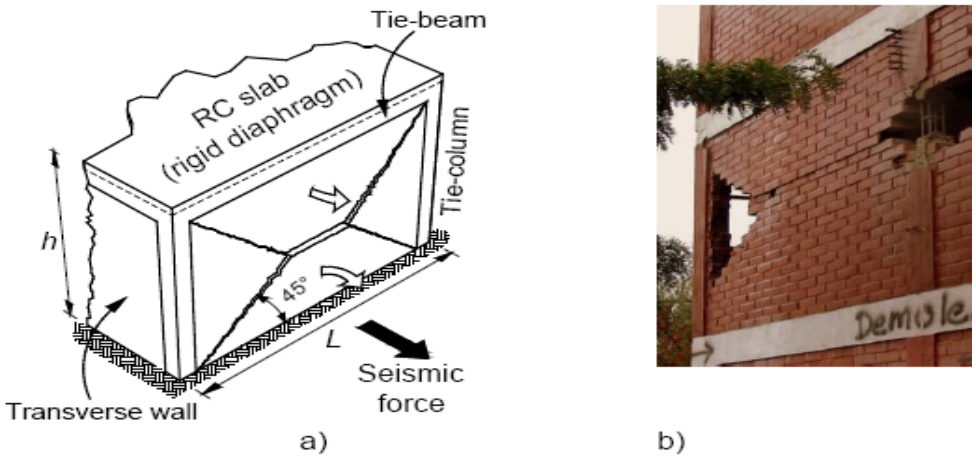


Figure 2.20: Out-of-plane seismic affects in confined masonry walls a) two-way slab mechanism, b) evidence from the 2010 Maule, Chile earthquake (Brzev et al, 2010).

RC tie-beams have an important role in enhancing the out-of-plane resistance of confined masonry walls in buildings with flexible diaphragms. These beams need to have adequate size and reinforcement (in terms of amount and detailing), as discussed in guideline of chapter four, section 4.5.1.3.2 and 4.5.1.3.3. Failure of a freestanding confined masonry fence in Santa Cruz due to the 2010 Maule, Chile earthquake is a good example of the out-of-plane wall collapse due to inadequate size of RC tie-beams and inadequate lap splice length, as shown in figure 2.21.

Figure 2.21: Collapse of a confined masonry fence in the 2010 Maule, Chile earthquake due to out-of- plane seismic effects: a) collapsed fence showing the RC tie-beam failure and b) details of RC tie-beam is showing excessively short lap splice length in the longitudinal reinforcement (Brzev et al, 2010).

2.10 Remedial Measures of Confined infilled frame failure due to earthquake

To prevent in-plane shear failure of infill wall, wall must be adequately confined and good proportion of mortar has to use in masonry bonding to increase shear strength. Also bricks quality has to be good. Adequate number of confining elements like tie column and tie beams, lintel should have to provide all around the opening as per code. Failure of tie column could take place either due to the flexural failure mechanism or shear failure mechanism. It has been observed from the past earthquake that the number of ties at the tie-beam-to-tie-column joint, and the detailing of the longitudinal reinforcement appear to play a role in the tie-column shear resistance. Buckling of longitudinal reinforcement was observed when size and/or spacing of ties at the ends of tie columns were inadequate (or when the crushing of the masonry units took place). So closed hoops or tie should have to provide near and among beam

column joint, also proper seismic detailing as per local code (chapter four, section 4.5.1.3.3) have to be followed.

To prevent the out-of-plane wall failure, it is important to restrict the maximum spacing of tie-beams and tie-columns (as per eurocode8, spacing of confining elements should be not more than 4 m) and ensure toothing/interaction between the walls and the confining elements. The eurocode8 also recommends limiting the out-of-plane failure of infill wall by limiting the slenderness ratio of wall to 15 (slenderness ratio is the ratio of effective height or lengths to thickness of the wall whichever gives more). Proper wall density should have to ensure to resist out-of-plane failure. The RC roofs and floors, which are rigid and flat and are bonded or tied to the masonry, have a positive effect on the wall, such as the slab or slab and beam construction be directly cast over the walls or jack arch floors or roofs provided with horizontal ties and laid over the masonry walls through good quality mortar, thus the rigid diaphragm easily transfer the seismic load to the wall. RC tie-beams have an important role in enhancing the out-of-plane resistance of confined masonry walls in buildings with flexible diaphragms. These beams need to have adequate size and reinforcement (in terms of amount and detailing), as discussed in section 4.5.1.3.2 and 4.5.1.3.3.

The following guideline has been suggested by Build Change (2011) to limit the out-of-plane failure of confined masonry building:

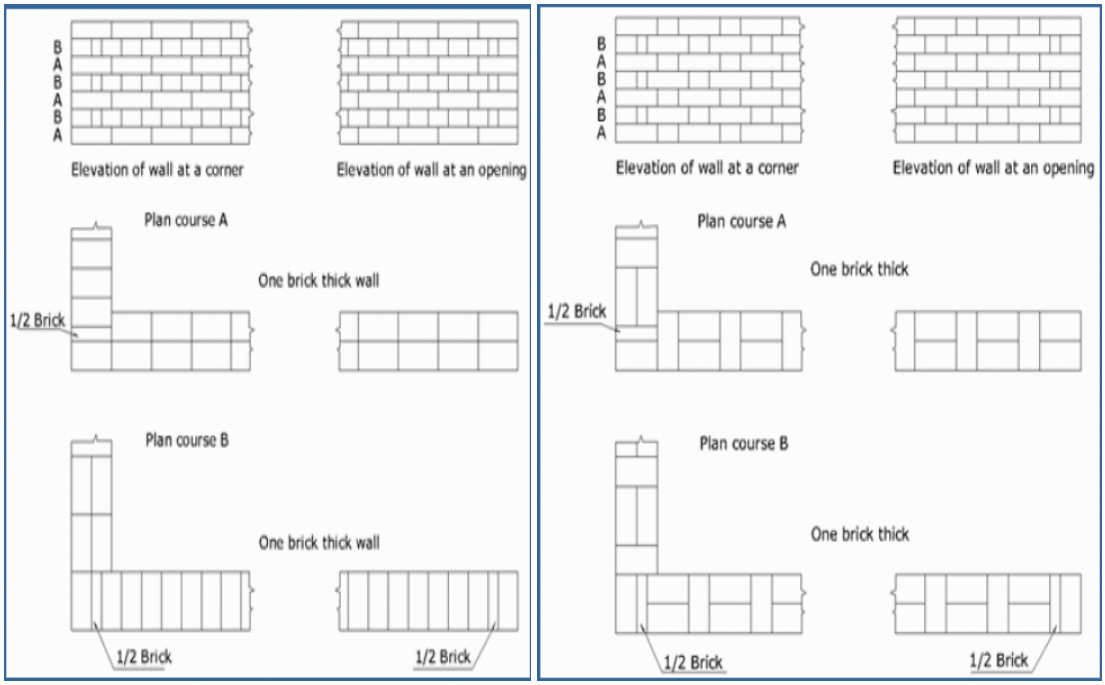
i) Follow configuration rules and add bracing at the ring/floor beam level for walls span of 4 m or greater and add cross walls every at maximum spacing of 4 m.
ii) Increase number and length of masonry shear wall in both directions.
iii) Restrain the masonry by tying it to the tie columns and bond beams or covering it with mesh.
    a) Add horizontal reinforcement in the masonry wall, tied into tie columns.

b) Lay wire mesh across wall when plastered must be on both sides, tied into foundation beam and ring beam, and have loops or ties through the brick wall.

iv) Reduce slenderness ratio h/t by using different Masonry Bond

a) Use a different masonry bond (English or Flemish) to make a wider wall but heavier wall will be more likely to crack and more deadly if it collapses as per figure 2.22.

b) Add a second horizontal bond beam (lintel level) must be at least 0.5 m between top of lintel tie beam and bottom of ring/floor beam for any gain of extra stiffness.



a) English Bond b) Flemish Bond

Figure 2.22: Masonry Bond preferred in confined masonry as per Eurocode8

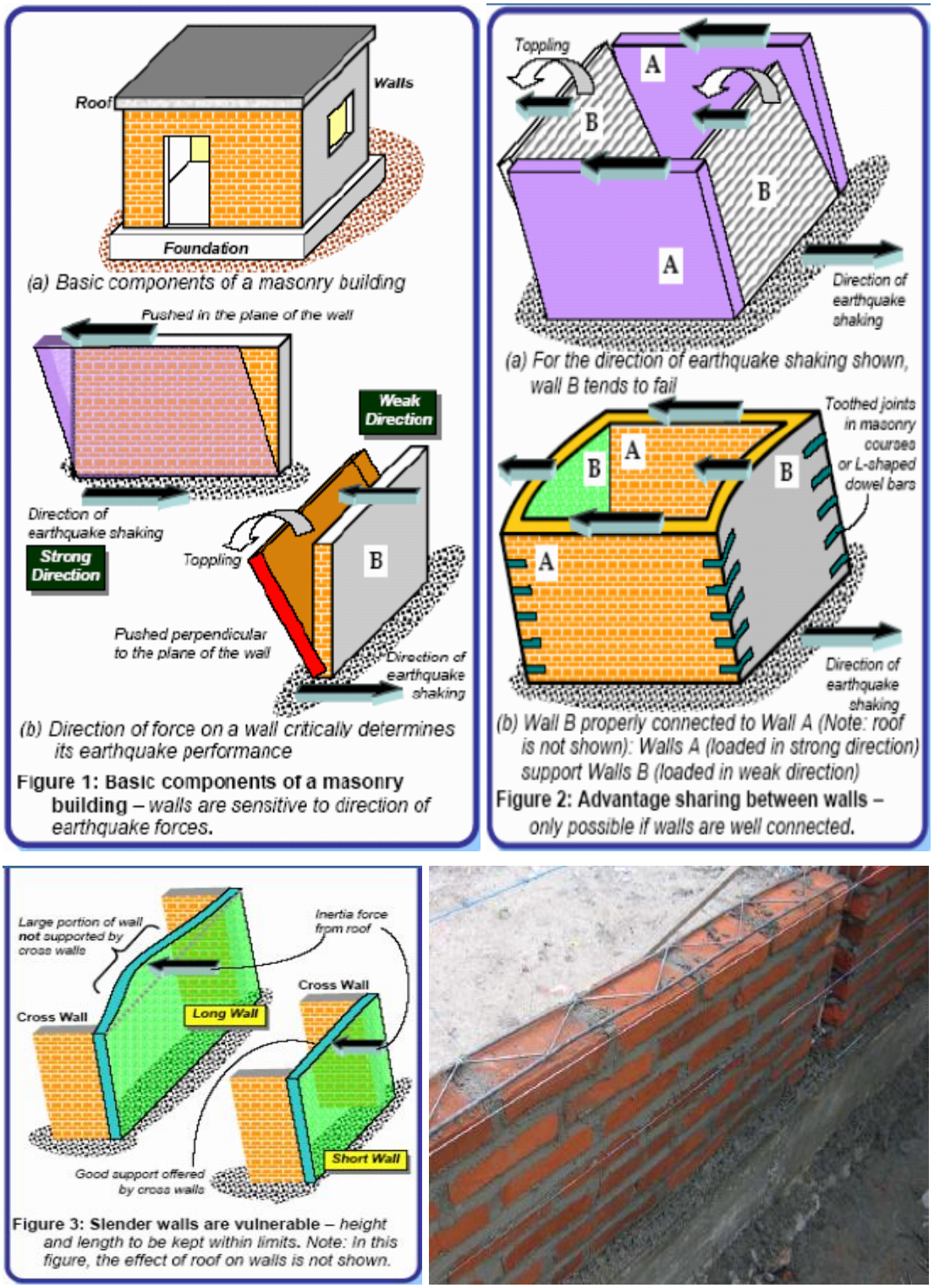


Figure 2.23: Remedial Measures of out-of-plane failure of infilled frame structure.

In an earthquake, the most vulnerable part of the single story confined masonry buildings is the masonry in the gable (above the ring/floor beam) due to very high slenderness ratio, high center of gravity (tendency to overturn), very difficult to

confine properly (poor connections between RC elements). To prevent collapse of masonry in gable it is advisable to not use masonry above ring beam without RC confining element. Also steel bars in center column should be developed in gable beam.

Even well designed earthquake-resistant buildings suffer some structural damage in moderate to large earthquakes. So one must design and detail buildings to ensure they do not collapse even when damaged. To design buildings for no damage in large earthquakes would be very expensive and impractical. In order to survive a moderate to strong earthquake a building must be:

• Configured well

• Detailed well

• Constructed well.

All three are important. If a building is poorly configured this increases the strength demands on structural members and exposes any weaknesses in detailing or construction quality. Building codes tend to concentrate on detailing; the designer must ensure the configuration is satisfactory. Construction team has to confirm strength of construction. Also care must be taken that all materials used are of good quality, and are protected from rain, sun, insects and other weakening actions, so that their strength lasts. Among all confined method of construction is the good system of lateral load bearing than bare frame only.

2.11 Advantage of Confined masonry construction over conventional RC bare frame

The advantages of using confined masonry over conventional RC frame construction for low rise building are both technical and economic in nature. For example,

- Vulnerability: An RC frame structure with masonry infill is more likely to cause deaths and injuries in a strong earthquake than a confined masonry structure. The masonry infill panel in an RC frame structure is vulnerable to collapse in an earthquake. In a confined masonry structure, the masonry infill can be not only

properly restrained from failure, but also leveraged to add strength to the structural system. Consideration of infill brick wall in analysis and design of RC frame structured building and adopting confined method of construction increases lateral resistance, storey shear capacity, lateral stiffness, energy dissipation capacity, improve ductility behavior and decreases storey displacement of the structure rather than conventional bare frame method. So confined masonry construction is more seismic resistant than conventional bare frame method.

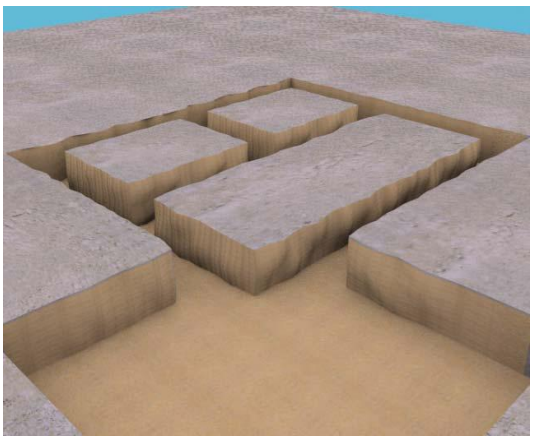
- Skill Level of Builders: A lower degree of skill, construction supervision, and tools are needed to build confined masonry houses than RC frame with masonry infill due to easy of construction.

- Cost: It is potentially less expensive to build a confined masonry structure than RC frame. As mentioned previously, smaller member sections, smaller diameter bars and fewer stirrups can be used in the tie beams and tie columns which are confining masonry walls.

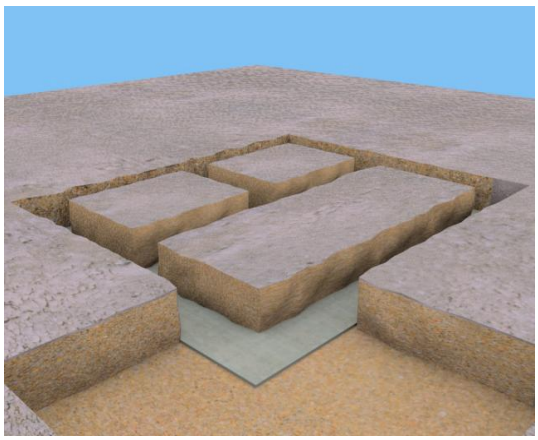
- Foundation: Adopting confined continuous masonry wall footing reduces foundation cost as well as increase resistance to differential settlement, tilting of foundation rather than isolated footing commonly provided in RC bare frame.

- Long Term Impact: It is more common in Bangladesh to build conventional RC bare frame building. But confined masonry construction is popular in our near country like Pakistan, India, Nepal and Bhutan and also in Europe, north and South America. So we can adopt confined masonry construction in low rise building design and construction to make the country's economy stable, strong by saving cost, by reducing seismic hazards, post earthquake risk mitigation. Working together with developers and homeowners to build with skills, materials, and tools they are likely to use in the future will have a longer-term impact on reducing deaths and injuries during earthquakes. The author hopes that, next revision of present national code BNBC-1993 will include seismic design guideline of confined masonry construction.

2.12 Confined Masonry Construction Sequences/Steps



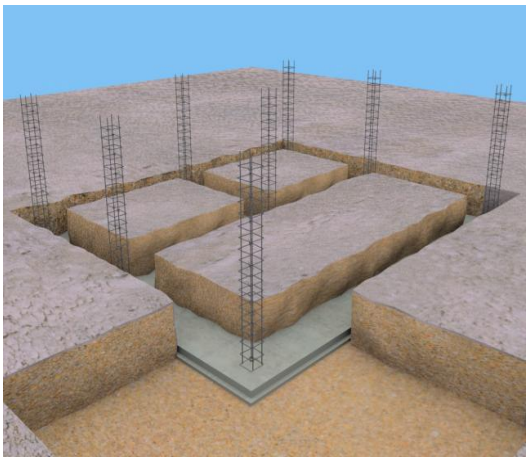
a)



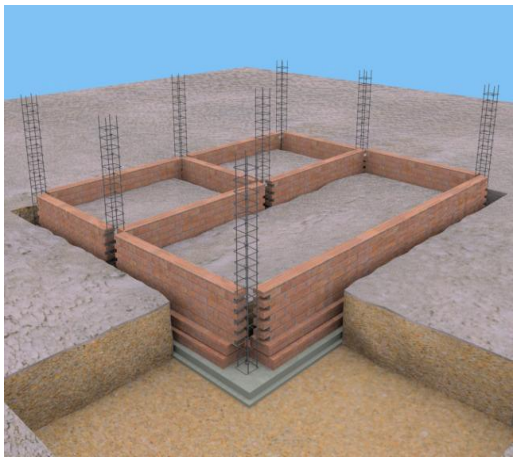
b)

a) Level the bed of the foundation after excavation.

b) Pour 1:3:6 plain cement concrete (PCC) or brick flat soling at the bottom of the foundation. The thickness of the concrete should be 75 mm and it should have a level surface.



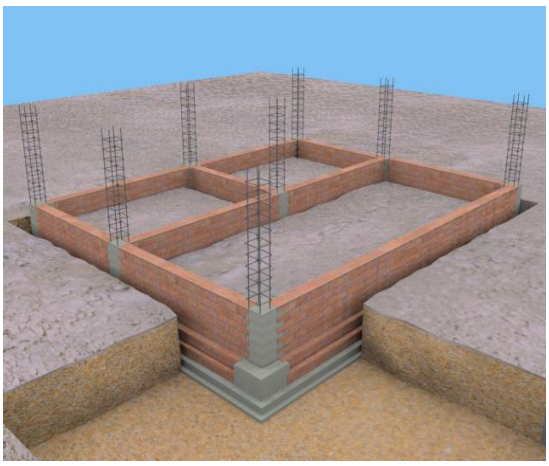
c)



d)

c) After tying the reinforcement of columns and foundation, pour designed strength concrete in the foundation having a thickness of 150 mm minimum. Due care must be taken while tying the reinforcement of the columns to ensure that the lap is provided at the middle of the first storey.

d) Construct the walls up to the plinth level and provide gaps at the locations of the columns.



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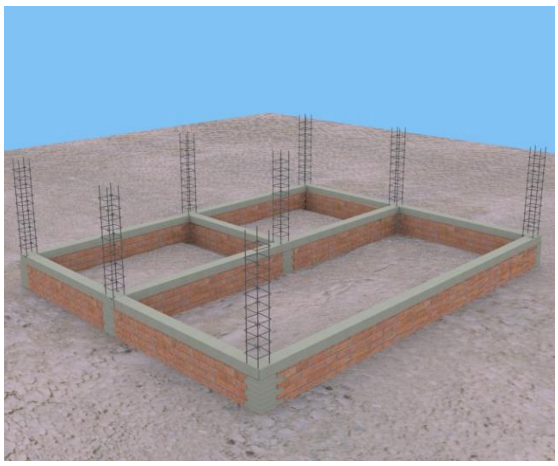
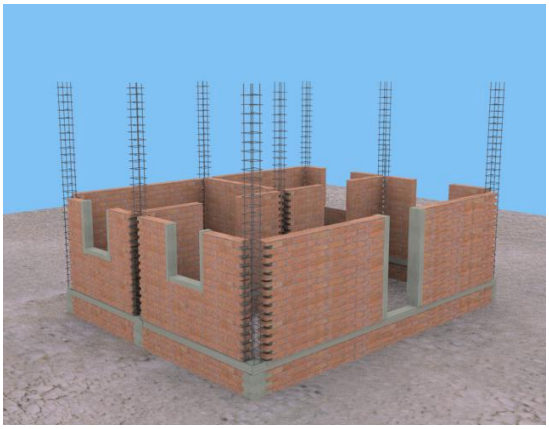


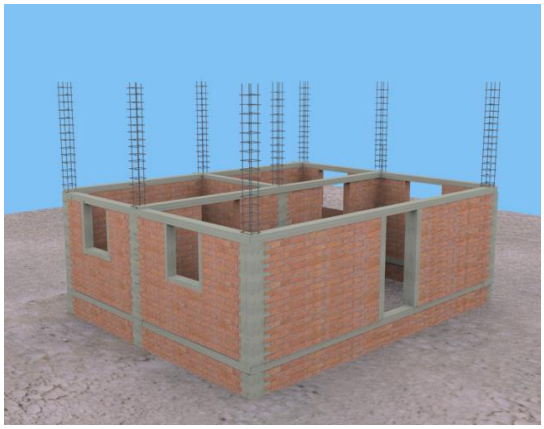
f)

e) Pour concrete in the columns up to the plinth level (below grade beam).

f) After tying the reinforcement of the grade beams, pour concrete in the grade beams.



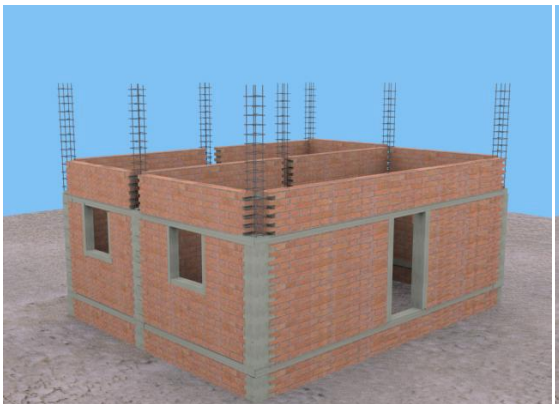
g)

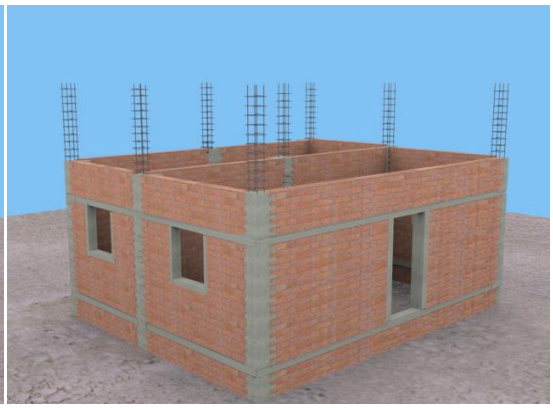


h)

g) Construct the wall up to the lintel level while leaving gaps for the columns. Extend the reinforcement of the columns further. In case of construction of second storey in future, the length of the column reinforcement should be such that to ensure a lap at the middle of the second storey.

h) Pour concrete up to the lintel level and in lintel.

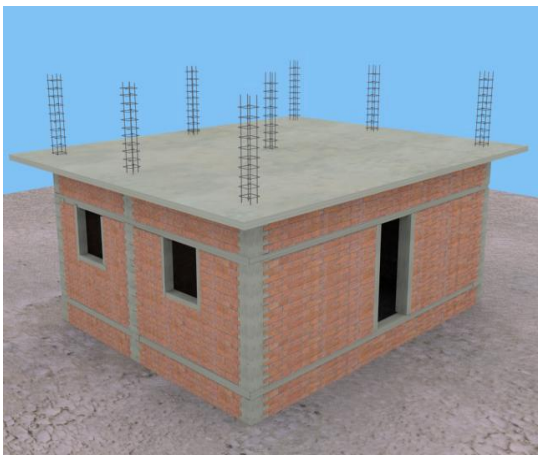


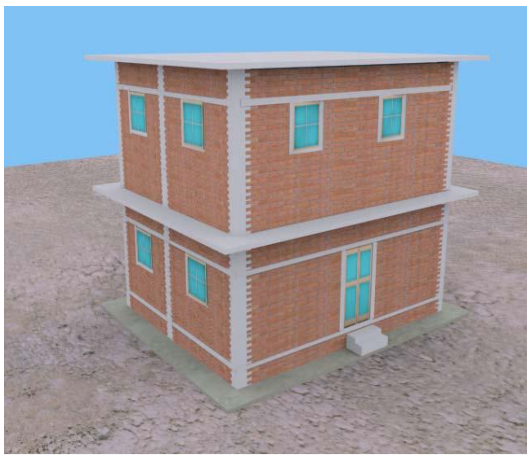


i) j)

i) Construct the walls up to the roof level and leave gaps for the columns.

j) Pour concrete in the columns up to the roof level.





k) l)

k) After tying the reinforcement of the slab, beams pour the slab concrete.

l) Construct the second and forwarding storey similar to the first story in a step-wise procedure.

2.13 Consideration of infill in current Codes

Most of the seismic codes ignore infill due to the brittle nature of failure, varying properties and low deformation capacity. However the presence of infill changes the behavior of structural system from frame action to truss action (Murty, 2000) due to

significant contribution of initial lateral stiffness. Some of the codes which consider the infill for seismic resistance are given below.

The BNBC (1993) considers the effect of infill masonry wall in terms of fundamental period (T) of vibration in section 2.5.6.2.a. which is based on effective cross sectional area of infill wall in ground floor only and response modification factor (R) as per table 6.2.24. The code does not provide specific guideline to design and construction and seismic detailing of confined masonry where confinement is in the form of tie column, tie beams and lintels. Some provision has stated in section 4.4, 4.5, 4.6, part-6 for the design of unreinforced and reinforced masonry but not for confined masonry construction. It is not possible to compute actual stiffness of the confined infilled structure due to absence of infill model generation methods in the code. The code also did not include percentage of loads carried by frame and masonry infill wall. Some guideline for compressive strength of masonry, mortar and modulus of elasticity was stated in section 4.3.2 and 4.3.8, part-6. Remedial measures for soft storey problem associated with infilled frame were not addressed in the code. The inter storey drift ratio is limited to 0.005 for T>0.7 seconds and 0.0025 for unreinforced masonry irrespective of consideration of infill wall confined in RC frame.

The IS1893 (2002) which is also used in Bhutan, consider the effect of infill masonry wall in terms of fundamental period of vibration. However there is no information on the basis of equation as it is empirically related to the height and width of the structure. Also same empirical equation is used irrespective of the extent of infill present in the structure. Moreover there is no control over choice of infill materials, giving wide option to the builders to select materials whose performance during earthquake is uncertain. As a result infill is considered as non structural components of building although literature revealed that there is a significant influence of infill wall on the lateral strength and stiffness of the structure. The soft storey problem associated with infilled structure is addressed by providing a prescribed magnification factor on structural member forces. It is not possible to compute actual stiffness of the confined infilled structure due to absence of infill model generation methods in the code. The inter storey drift ratio is limited to 0.004 irrespective of consideration of infill wall confined in RC frame.

Eurocode8 (2003) considers the effect of infill masonry wall on the fundamental period of vibration taking into a correction factor (C_t) derived based on effective cross sectional area of infill wall in ground floor. It consider the masonry infilled RC frames as 'dual' structural system which are classified into three ductility classes, namely, high, medium and low. The effect of infill is neglected in low ductility class. It requires the frame members to resist 100% of the vertical load and 50% to 60% of the total horizontal loads on the structure. This code allows reasonable irregularities in plan by doubling the accidental eccentricity but recommend dynamic analysis for an unacceptable irregularity problem. It recommends that the infill wall which has only one opening, either door or window, has a significant influence on the frame. For other walls which have more than one opening, proper measures such as reinforcing the wall and providing concrete member along the perimeter of the opening are recommended. The code also recommends limiting the out-of-plane failure of infill wall by limiting the slenderness ratio of wall to 15. Slenderness ratio is the ratio of effective height or lengths to thickness of the wall whichever gives more. The stiffness of infill wall is taken into consideration by recommending the use of diagonal strut. However the thickness of diagonal strut is not specified as it varies with the opening. It clearly mentions the seismic design principal and seismic detailing of confined masonry. There is no mention of modulus of elasticity of infill materials.

Nepal Building code (NBC-201, 1995) considers infill by recommending the use of pin jointed diagonal strut elements as an infill wall. However the width of diagonal strut is not recommended and hence the consideration of opening is not realized. The distribution of axial forces and lateral seismic loads are specified. The code also recommends the Young’s modulus of infill materials to be 2500 to 3000 MPa. The walls which have opening less than 10% of the wall area is considered as structural walls and if the opening exceeds 10%, the wall are provided with reinforced concrete elements all around the opening perimeter and recommend appropriate reinforcement. The out-of-plane failure is prevented by providing the concrete bands at one third and at two third of the wall height. However there is the code which recommends the isolation of infill wall from the frame (NZS-3101, 1995) to ensure ductile behavior of RC frame.

2.14 Recent Research of Confined Masonry Infill Structure

A number of experimental and analytical studies were performed to evaluate the effect of masonry infill panels on the seismic behavior of frame structures. Polyakov (1960) conducted experimental tests on masonry-infilled frames, first proposing that the infill system works as a braced frame, with the wall forming compression “struts”. Following this approach, Stafford-Smith (1962) and Mainstone (1971), among others proposed methods for calculating the effective width of the diagonal strut, supported by test results from mortar panels and infilled frames, respectively. Other experiments examined the performance of infilled frame structures more broadly. Klingner and Bertero (1978) tested a one-third scale 3.5 story representation of an 11-story 1970s-era RC apartment building. Their study concluded that reinforced infill panels reduce the risk of incremental collapse, compared to a bare RC frame. Mehrabi et al. (1996) tested twelve ½-scale single-story single-bay frame specimens and observed that the frames with infill showed better seismic performance than the bare frames. Flanagan and Bennett (1999) used a piecewise-linear equivalent strut to model infill and proposed an analytical procedure to calculate the strength of the infill, based on experimental results of 21 steel frames with clay tile infill walls. Other researchers have used finite-element models to represent complex aspects of wall behavior.

Dhanaskar and Page (1986) modeled an infilled frame using nonlinear finite element brick elements, comparing the results with several half-scale experiments. Mehrabi and Shing (1997) used a smeared-crack finite element model to represent masonry units and RC frames, developing a constitutive model for mortar joints. Dymiotis et al. (2001) assessed the seismic vulnerability of a 10-story infilled RC frame at “serviceability” and “ultimate” limit states. Amanat (2006) studied the rational for determining the natural period of infilled RC frame using FE models. The important observation was that, the amount of infill has significant influence on the fundamental period of the structure but the distribution of infill within the structure has no significance influence. This reports recommended the magnification factor for the parameters such as span length, number of span, and the amount of infill to modify fundamental period of bare frame given by code equation. Joao (2007) carried out sixteen tests in unreinforced, confined and reinforced masonry walls to study the

influence of confinement and horizontal reinforcement in masonry shear walls subjected to cyclic loading. Dolsek and Fajfar (2008) used concentrated plasticity beam-column model elements with equivalent strut wall elements to evaluate the seismic performance of masonry-infilled RC frames, looking at “damage limitation”, “significant damage” and “near collapse” limits states. Madan and Hashmi (2008) evaluated the performance of 7 and 14-story RC frames with masonry infill subjected to near-fault ground motions. Stavridis and Shing (2009) have developed a complex nonlinear finite element model for RC frames with masonry infill, combining the smeared and discrete crack approaches to capture different failure modes observed in experiments. More recent research has combined analytical and experimental methods to evaluate the seismic performance of RC frames with masonry infill more generally. Agha et al. (2010) conducted experimental program and studied the seismic behavior of four half scale perforated brick masonry walls under cyclic loading. Test result shows that the confining elements like tie column increased the shear capacity and displacement ability, and improved the ductility and dissipated energy of confined masonry (CM) walls significantly.

Dorji (2009) studied the seismic performance of brick infilled RC frame structures in low and medium rise buildings in Bhutan. It was reported that the effect of infill wall thickness does not have influence on global response of structure under seismic action, except moments and shear forces. Infill wall thickness exerts very little affects on inter-storey drift, stresses in infill wall, roof displacement and fundamental period, so thereby insignificant. It was stated that, for 125 mm and 250 mm thick infill wall, the difference of fundamental period is 1.4%, roof displacement and inter storey drift ratio increases 4.69% and 1.15% respectively. It was also reported that, for 125 mm and 250 mm thick infill wall, moments in the columns and beams were increases by 9.42% and 9.54%, shear force increased by 21.06% and 18.37% in the column and beam respectively. Parameters of Kobe earthquake like PGA=0.2g, 5% damping was used in non linear time history analysis.

2.15 Performance in recent Earthquake

In February 27, 2010 Maule, Chile earthquake (M_w=8.8) low rise confined masonry buildings performed very well. Figure 2.24 shows a two story confined masonry house in Curepto, Maule which remained virtually undamaged, while the adjacent adobe house has collapsed (Astroza and Brzev, 2010; Seismic design guide for low rise confined masonry buildings, August 2011). 521 People dead due to conventional bare framed building failures whereas only 10 dead due to confined masonry construction in this earthquake (www.confinedmasonry.org).

Fig 2.24: Performance of confined masonry building in recent significant earthquakes: The 2010 Maule, Chile earthquake. (Brzev, Astroza and Moroni, 2010)

Most one and two storey single-family confined masonry dwellings did not experience any damage, except for a few buildings which suffered only a moderate damage. A similar statement can be made for three- and four-storey confined masonry buildings, where the large majority of buildings remained undamaged, with only a few buildings having suffered severe damage, and two three-storey buildings which collapsed. An investigation study shows that confined masonry building fails due to (1) inadequate wall density of 0.5 to 0.7% only (2) Inadequate quality of masonry

materials and construction (3) Deficiencies in detailing of reinforcement in RC confining elements as per code (4) Absence of RC confining elements at openings (5) Geotechnical issues like steep slope behind building (Brzev, Astroza and Yadlin, 2010).

Fig 2.25: Performance of confined masonry building in recent significant earthquakes: the 2007 Pisco, Peru earthquake. (www.confinedmasonry.org)

A very similar observation was made after the 2007 Pisco, Peru earthquake (M_w=8.0), where confined masonry buildings performed very well compared to other types of masonry and conventional frame buildings which were badly damaged or collapsed. Figure 2.25 shows a four-story confined masonry building in Ica, Peru which remained virtually undamaged in the earthquake (Seismic design guide for low rise confined masonry buildings, August 2011).

Central Java, Indonesia M 6.3 earthquake on 27 May 2006 was the most deadly event killing 5,782 people, had the most devastating effect on housing stock, damaging or destroying 135,000 houses, and yielded compelling examples of good performance of confined masonry houses in villages where 70-90% of the other buildings were destroyed or heavily damaged (HAUSLER, 2008).

Fig 2.26: Performance of confined masonry buildings in recent significant Earthquake: the 2006 Central Java, Indonesia earthquake. (www.buildchange.org.)

Many newly built confined masonry houses with reinforced concrete tie columns and bond beams at the plinth and roof levels performed well in this earthquake. Figure 2.26 shows a well-built confined masonry house with no evidence of damage, on the edge of heavily damaged Pellet in 2006 Central Java, Indonesia earthquake.

From the records of past earthquake, it has been clear that confined masonry building performed well rather than conventional bare frame during earthquake.
As per "seismic design guide for low rise confined masonry buildings" August, 2011 proposed by confined masonry network, wall density index, d should be minimum 2% for soft clay soil in moderate seismic zone to resist the collapse of confined masonry building during earthquake. In this projects works wall density index, d for confined masonry building is 3.87%.

2.16 Guideline of Confined Masonry design and Construction

Based on the present study guidelines for the confined masonry construction can be broadly discussed in the stages of design process and construction procedure that can be applicable for Bangladesh in the following section.

2.16.1 Design Process

Finite element method is the best method for modeling confined masonry infill wall in RC frame structured building as it is similar of real structural existence and it shows more accurate numerical and experimental results. Simple diagonal strut method allows various codes but it shows less accurate results in comparison with Finite element method. Linear static analysis can be done for the structure having no in-plane irregularity, out-of-plane irregularity, severe weak story irregularity and tensional irregularity as per FEMA 356, but nonlinear dynamic analysis is required for this type of stiffness irregularity.

The following guideline can be followed prior to and during design of confined masonry RC frame building.

2.16.1.1 Guideline of Foundation

For the purpose of making a building truly earthquake resistant, it will be necessary to Choose an appropriate foundation type for it. Since loads from typical low height Buildings will be light, providing the required bearing area will not usually be a problem.

- The foundation design is based on several requirements. Two of the mains considerations are that (1) settlement due to the building load must not exceed tolerable values and (2) there must be an adequate factor of safety against a bearing capacity failure. In most cases, settlement governs and the foundation bearing pressures recommended by the geotechnical engineer are based on limiting the amount of expected settlement due to the static or seismic cases. In other cases where settlement is too high, the building is often constructed with a deep foundation, which also reduces the possibility of a bearing capacity failure.
- To determine the allowable bearing pressure q_{all}, the ultimate bearing capacity q_{ult}, is divided by a factor of safety. The normal factor of safety used for static bearing capacity analyses is 3. For the evaluation of the bearing capacity for seismic analysis, the factor of safety is often in the range of 5 to 10. These are high factor of safety compared to other factors of safety, such as only 1.5 for slope stability analyses.

- Building codes often require minimum footing sizes and embedment depths. Larger footing sizes will lower the bearing pressures on the soil and reduce the potential for static or seismic bearing capacity failures.
- A common problem in foundation design is the overestimation of the shear strength of the underlying soil. Another common situation leading to a bearing capacity failure is the loss of shear strength during the earthquake, because of the liquefaction of the soil or the loss of shear strength for sensitive clays. So this situation has to analyze before foundation selection.
- In firm soil individual column footing can be provided. But in soft soil footings shall be continuous below all walls i.e. continuous wall footing.
- The depth of footing in the soil should go below the zone of deep freezing in cold countries and below the level of shrinkage cracks in clayey soils, also all loose soil have to remove before selecting foundation bottom level. The bottom of footing shall be at least 1.5 m below grade level.
- Slope the sides of foundations trenches approximately 2:1 (rise to run) to maintain Stability.
- Width of concrete strip footing below all walls shall be at least 0.5 m for single story houses. For two or three stories provide footing width as per calculation.
- Use Minimum 250 mm thick masonry wall as foundation wall with water proofing plaster of 12 mm thick.
- Extend reinforced concrete column to concrete footing. Build masonry foundation wall prior to casting concrete column extensions.
- Provide closely spaced steel ties in column at 125 mm spacing between Grade beam and concrete footing.
- Compact the backfill around foundation wall and footing by sand.
-

2.16.1.2 Masonry Wall

2.16.1.2.1 Wall density requirements

Wall density is the key parameter influencing the seismic performance of confined masonry buildings. Evidence from past earthquakes show that confined masonry

buildings with adequate wall density were able to resist the effects of major earthquakes without collapse.

The wall density is quantified through the wall density index, d, which is equal to

d = AW/AP

Where

AP is area of the building floor plan, and
AW is equal to the cross-sectional area of all walls in one direction.

It is very important to note that wall cross-sectional area should not be included in the Aw calculation in the following cases:

a) Walls with openings, in which the unconfined opening area is greater than 10% of the wall surface area, and

b) Walls characterized by the height-to-length ratio greater than 1.5.

For Soft clay soil or soft sand and compact granular soil in moderate seismic zone, wall density index should not be less than 2% as per eurocode8 (2003).

2.16.1.2.2 Opening in wall

Presence of significant openings has a negative influence on seismic resistance of confined masonry walls, according to research evidence and reports from past earthquakes. Ideally, confining elements (RC tie-columns) should be provided on the sides of the openings, but that is not always feasible.

- As per eurocode8 opening in wall less than or equal to 16 sft area can be provided in confined building without any additional confining elements like bands or tie columns.
- Windows without confining elements are permitted only when centered on a wall panel and when one of the following is satisfied: 1) the window opening is the only window opening located on that side of the building; 2) all openings adjacent to the window opening are surrounded by confining elements.
- For wall opening more than 16 sft and if confining elements (RC tie-columns) are not provided at the openings, then the panel is not considered to be confined. As a result, the panel should not be considered in wall density calculations and its contribution to seismic resistance of the building should be disregarded.

- When an opening is located at the intersection of the panel diagonals the panel cross-sectional area (AT) considered in wall density calculations should exclude the opening length.
- The maximum window opening height for unconfined windows shall be the minimum of 33% of the wall height including the confining elements.
- Door openings shall be a maximum of 1m wide.
- Tie columns shall be located on either side of all door openings if possible.
- It is not permitted to locate door openings adjacent to orthogonal walls or wall corners for single-story houses with lightweight roofs without tie column.
- Lintel should be provided above any kind and size of opening of equal to wall thickness, minimum depth=150 mm with stirrup 10 mm @150 mm c/c minimum.

Fig 2.27: Good lintel providing practice in confined masonry construction.

2.16.1.2.3 Wall spacing

Maximum spacing of transverse walls in confined buildings with flexible diaphragms should not exceed

- 6 m for regions of low and moderate seismicity, and
- 4.57 m for regions of high seismicity.

2.16.1.2.4 Wall dimensions and height/thickness ratio restrictions

- A minimum wall thickness (t) of 125 mm is required.
- The maximum wall height/thickness (H/t) ratio for walls in low rise confined buildings must not exceed 25.
- The height/length ratio of a wall panel should not be less than 0.5.
- The maximum wall height should not exceed 3.05 m.

2.16.1.2.5 Parapets

Tie-columns should extend to the top of the parapet. When a parapet is not confined, the height should not exceed 0.5 m, otherwise the height limit is 1.2 m.

2.16.1.2.6 Toothing at the wall to tie-column interface

Good bonding between a masonry wall and adjacent RC tie-columns is important for satisfactory earthquake performance, and for delaying undesirable cracking and separation at the wall-to-tie column interface. Bonding is an essential feature of confined masonry construction and it can be achieved by toothing provided at the wall-to-tie-column interface. Toothed edges should be left on each side of the wall at the interface with the tie-columns. Toothing length should be equal to one-quarter of the masonry unit length, but not less than 50 mm.

a) b)

Figure 2.28: Toothing applications: a) recommended construction practice and b) Not recommended - absence of toothing in concrete block construction

2.16.1.3 Confining/Frame Elements (tie Columns and tie Beams)

2.16.1.3.1 Spacing

Tie-columns:

Tie-columns should be provided at the following locations:

- at wall intersections, and
- at ends of wall panels that provide lateral load resistance to the building.

Spacing of tie-columns should not exceed:

- 4.57 m for regions of high seismicity, and
- 6 m for regions of moderate and low seismicity.

Tie-beams (sometimes known as Ring beam):

A RC tie-beam must be provided at the top of each wall at the maximum. Provision of continuous RC tie-beams at intermediate (lintel/sill) levels is not necessary, but it may be beneficial for out-of-plane stability of slender walls characterized with large height/thickness ratio (greater than 20).

2.16.1.3.2 Minimum dimensions

As per eurocode8 (2003),

- Tie-column size (depth x width): 150 mm x 150 mm
- Tie-beam size: same as tie-column size

But column size has to keep larger for strong column weaker beam concept.

2.16.1.3.3 Reinforcement requirements and seismic detailing

Longitudinal reinforcement (tie-beams and tie-columns):

As per eurocode8, number of tension reinforcement is not limited, but to maintain similarity with other code minimum 4 number bar have to use. The area of reinforcement in each vertical and horizontal confining element shall not be less than 240 mm^2. Also as per eurocode8 minimum diameter of bar is 5 mm for main and tie bar, but to maintain similarity with other code minimum 10 mm diameter deformed bar of minimum 60 grade have to use for both tie column and tie beams.

To ensure the effectiveness of tie-beams in resisting earthquake loads, longitudinal bars should have a 90° hooked anchorage at intersections. Proper detailing of tie-beam-to-tie-column connections is a must for satisfactory earthquake performance of the entire building. It is very important to ensure continuity of longitudinal tie-beam rebar through the joint. Note that tie-column reinforcement needs to be extended into the tie-beam as much as possible, preferably up to the underside of the top tie-beam reinforcement. In buildings with RC floors and roof, it is acceptable to integrate a RC tie-beam into RC slab.

When tie-beam depth exceeds 300 mm, vertical reinforcement in the RC tie-column must be confined by the ties, below and above the joint. An additional U-shaped stirrup must be placed at the tie-beam mid height. Additional confinement is not required for vertical reinforcement in joint regions of interior tie columns. However, to minimize the chances of buckling in vertical reinforcement it is recommended to place the first tie at the ends of tie-columns (top and bottom) as close to the joint as possible, this applies to all seismic regions.

General requirements for lap splices in longitudinal reinforcement are summarized below:

- The tie-beam reinforcing is hooked and lapped at the ends with the intersecting reinforcing. The lap length of the hook tails with the intersecting reinforcing should be at least 15 to 20 bar diameters or as specified on the drawings.
- Tie-column longitudinal bars at the roof level should be bent and lapped 40 diameters with the tie-beam reinforcing.
- Tie-column longitudinal bars at the lower floor levels should extend far enough above the floor slab to be able to lap splice at least 40 bar diameters with the tie-column bars to be placed above.
- Lap splices for longitudinal reinforcing should be at least 40 bar diameters. In tie-beams, the splices should be located at the end 1/3 length. The splices should be staggered so that no more than 2 bars are spliced at any one location. If the construction drawings specify 180 degree hooks at the bar ends this should be verified.

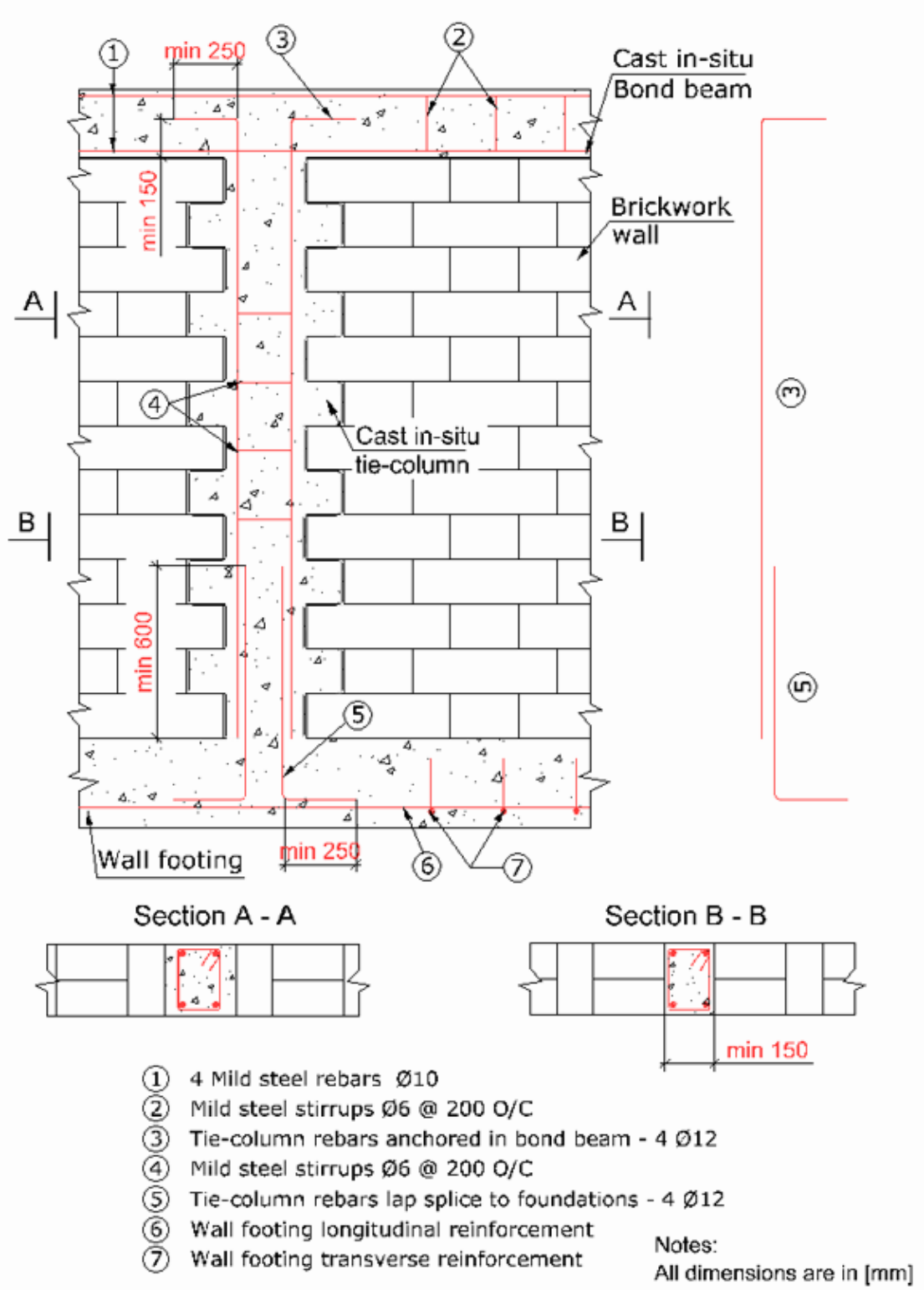


Fig 2.29 Reinforcement details for confined masonry construction according to the Eurocode8 seismic provisions;

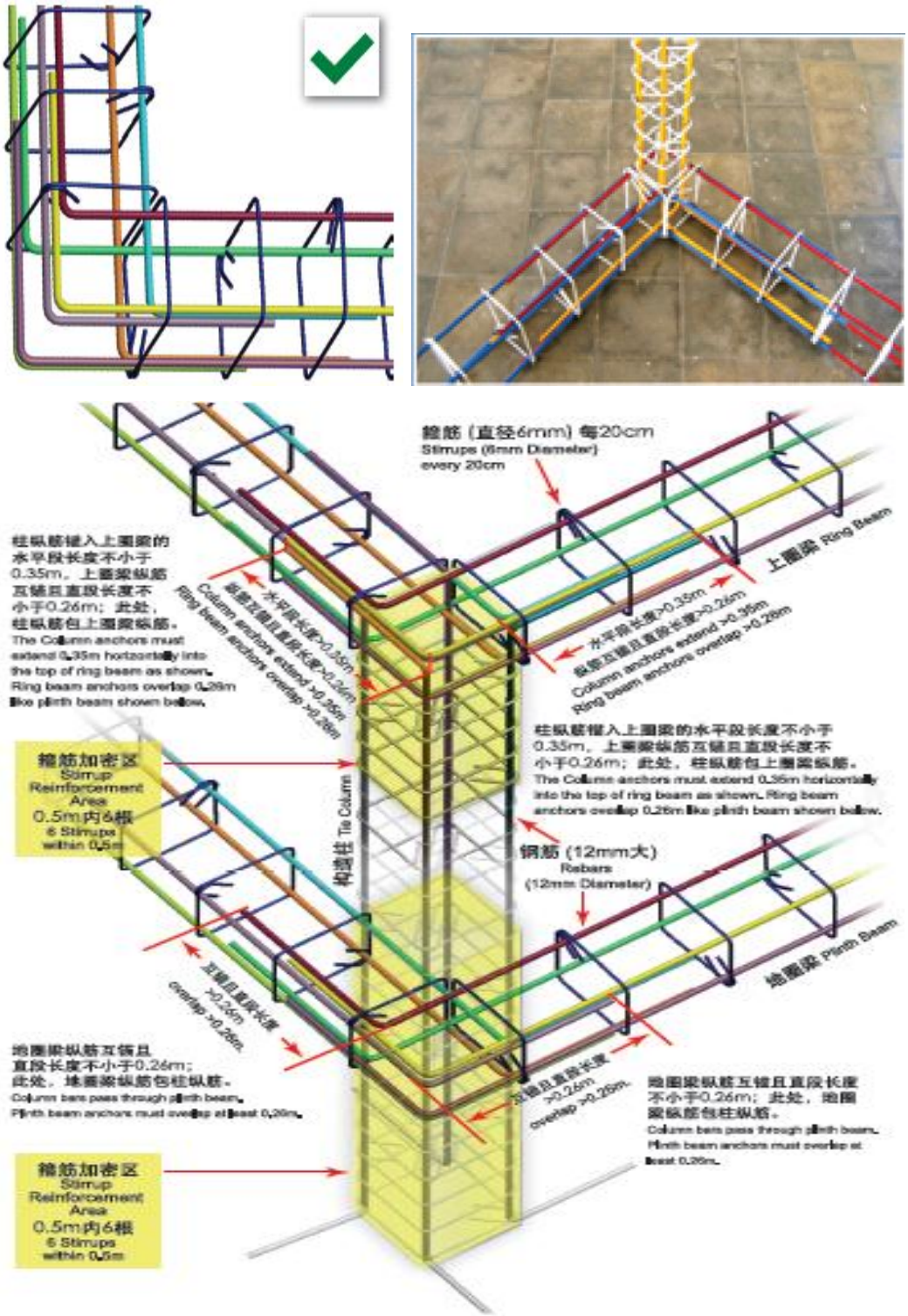


Figure 2.30: Joints details of confining column and beam (www.buildchange.org.)

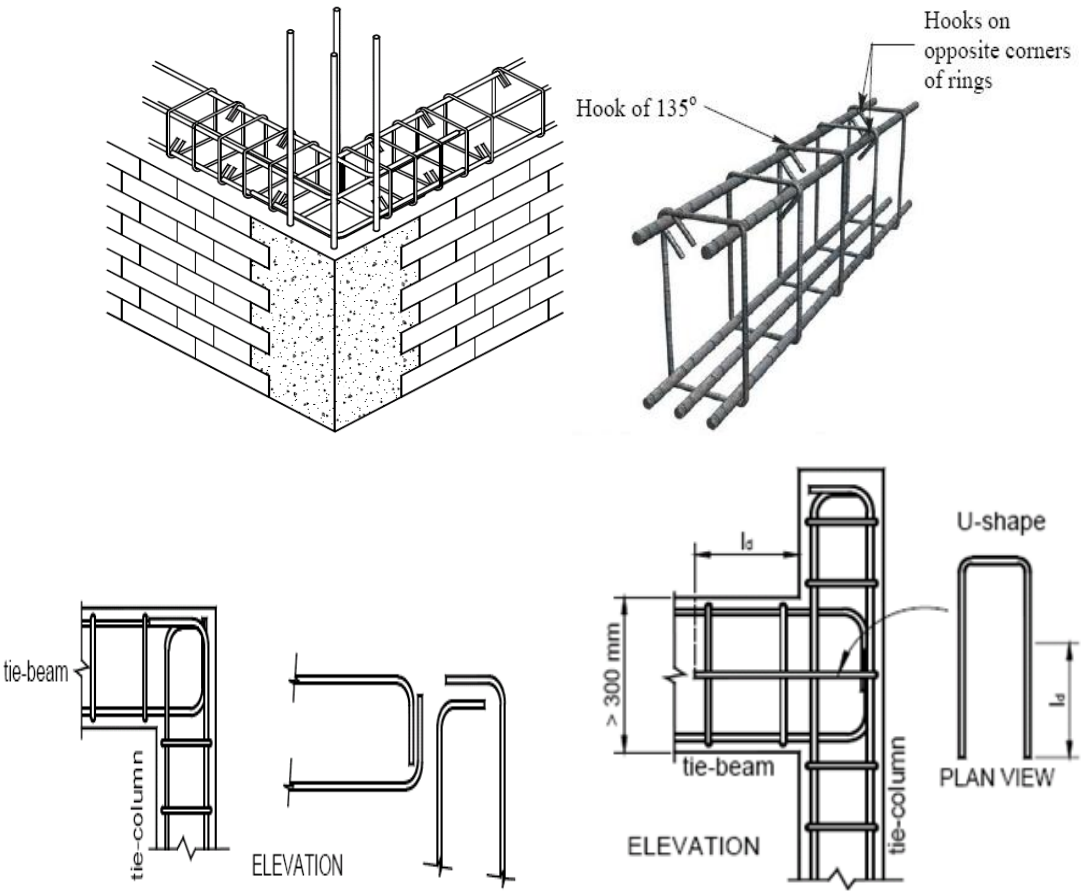


Figure 2.31: Anchorage of tie beam and tie column longitudinal reinforcement and additional confinement at joints (Alcocer et al., 2004)

<u>Ties or stirrup (tie-beams and tie-columns):</u>

- Size: minimum 5 mm diameter bars should be used as per eurocode8 (either smooth or deformed steel bars can be used) with 135° hooked ends (staggered). But 10 mm diameter bar to be used for better capacity.
- Spacing:
  - As per eurocode8 maximum tie spacing is 150 mm which is applicable for all seismic zones.
  - As per seismic design guideline for low rise confined building prepared by confined masonry network ,a Project of the World Housing Encyclopedia, EERI & IAEE in July 2011 Tie spacing (s) should not exceed 200 mm.
    a) For regions of high and very high seismicity, reduced tie spacing (s/2) is required at the ends of tie-columns. The length over which the

reduced tie spacing is used should not exceed the larger of the following two values:

- 2b, where b is the tie-column dimension, or
- h/6, where h is the tie-column clear height.

b) For regions of moderate and low seismicity, an uniform tie spacing (s) of 200 mm should be used throughout - it is not required to reduce tie Spacing at the tie-column ends.

- Minimum concrete cover to ties is 20 mm.

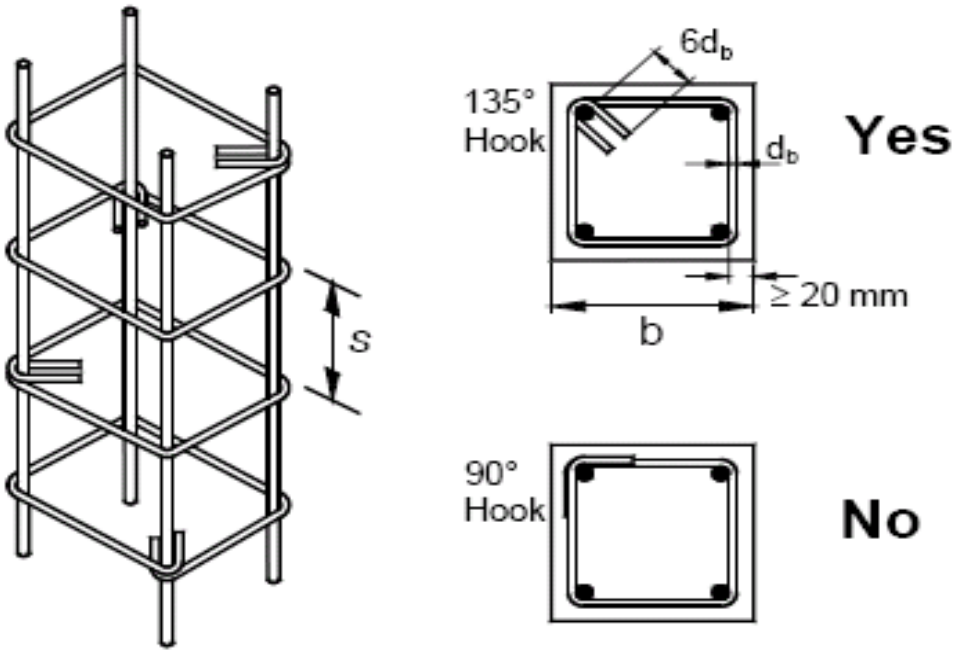


Figure 2.32: Hook details of confining column ((Alcocer et al., 2004)

2.16.2 Construction Procedure

Properly designed and built confined masonry buildings typically performed well in past earthquakes. Construction quality has a significant bearing on the seismic performance of confined masonry buildings. In general, it is highly desirable to ensure quality of construction by performing continuous inspection by qualified professionals. However, in rural areas building constructed by local mesons. The attempts of this study are to design a footing which can be constructed by local masons without difficulties.

2.16.2.1 Foundation and Grade Beam

The foundation should be constructed in the similar manner as in traditional masonry construction. Either an un-coursed random rubble stone masonry footing or an RC strip footing can be used. A RC plinth band/Grade beam should be constructed on top of the foundation. In confined masonry construction, a plinth band is essential to fully confine wall panels along their bases and to prevent excessive wall damage due to building settlement in soft soil areas. Note that the vertical reinforcement should be extended from a RC tie-column into the plinth band, and whenever possible, into the foundation. Concrete block masonry units can be used for foundation construction below the ground level - it is not recommended to use other masonry units for this purpose. A 3" thick brick flat soling can be provided below base for the leveling of foundation.

In RC strip footing, RCC base have to cast first by extending tie column rebar from base. Then 10" masonry brick wall have to bond with mortar ratio of 1:3 from base top to beneath grade beam keeping space for tie column. Cast tie column up to grade beam bottom. Masonry brickwall bonding works continue after casting grade beam at a time to confirm rigidity. After foundation concreting compact the backfill around foundation wall and footing by sand.

Digging the pit for the septic tank or underground water reservoir first so we could take a look at the soil profile and screen for liquefaction hazard and soft, expansive clays or peats. Although if the water table was within 2-4m of the ground surface, the soil was clayey, so liquefaction was not considered a hazard. Expansive clay was a bigger concern. Expansive clays were identified by touch and shrinkage tests. When it was encountered, we dug it out and replaced it with compacted fill of sand.

2.16.2.2 Super Structure Construction quality

2.16.2.2.1 Building Materials Quality

Bricks:

Fired clay bricks can be used as it is locally available and relatively cheap. The type of clay and the firing process had the biggest impact on brick quality.

Simple three-point bending tests in figure 4.20 ot "T" test can be done to ensure quality and the following checks to evaluate brick strength in the field

- No cracks or chips
- No visible unmixed portions or divits
- Brick is square, not warped or curved
- Dimensions are consistent among a sample of 10-20 bricks; they do not vary by more than 10 mm in the long direction and 5 mm in width and height
- When two bricks are hit together, the sound is a metallic clink, not a dull thud
- When left out in the rain or soaked in water for 24 hours, bricks do not crumble

Fig 2.33: Three point bending test for brick strength, use average size, no bouncing.

Cement:

Two types of cement are common in Bangladesh: Type 1 Portland Cement (BDS 197 CEM-I, ASTM C-150) and Portland Composite Cement (PCC) (BDS 197 CEM-II or ASTM C-595). We can use PCC for concrete foundation, floor, tie beam, tie column, the masonry wall and plaster, because of the increased workability and lower price.

Rebar:

All rebar have to free from rust or any other raw materials. It should be uniform color. Deformed bar of 40 grade can be used for slabs, water tank, septic tanks and grade 60 grade can be used for foundation, tie column, tie beam, stair etc. Verify grade of steel by checking marking on bars (Grade 40 or Grade 60). Do not reuse old or bent reinforcement.

Sand:

Like gravel, depending on the source, the sand was often mixed with fine clay particles. To evaluate sand in the field, we put a handful of sand in a plastic cup or bottle, filled it with water, and shook it up. If the water was clear, the sand was accepted. If it was cloudy, it was rejected. But a maximum 3% clay content can be allowable in sand. FM can be ensuring by laboratory test.

2.16.2.2.2 Concrete mixing and Pouring

Concrete can be prepared by mixing up ingredients at 1:2:3 by volume on the ground or on a paved surface. Builders had a tendency to add too much water to the mix, especially when using a mechanical mixer. Slump test can be done for consistency and simply picking up a handful of mixed concrete and if the water (and cement) ran out through one's fingers, it was too wet. So proper water cement ratio have to maintain to obtain desired strength and durability of concrete. Water quantity can be control by measuring water in a mark able jar or pot of 25 liter capacity.

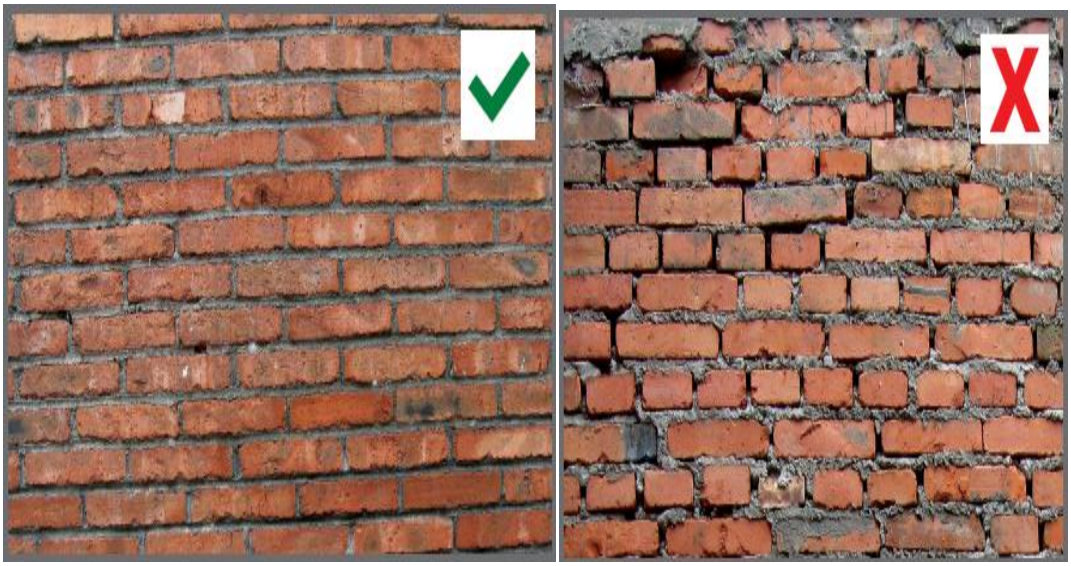
Concrete spacers were used to separate the steel from the formwork. Concrete spacers were known about but not common; if the builders used spacers, they used small stones rather than squares of concrete with binding. Formwork was wetted prior to pouring concrete. In the pilot project, rammed the concrete with a vibrator and tapped the formwork with a hammer in order to compact the concrete. Sometimes the

workers had a tendency to over vibrate and liquefy the concrete. So we have to careful about vibration. We required casting the entire Grade beam all in one day. Concrete was cured by sprinkling water or ponding or wrapping hatches cloth and polythene over it on concrete surface for minimum to seven days.

- For Foundation, beams, columns and roof slabs, use a 1:1.5:3 mix ratio (cement:sand:gravel) to achieve 3000 psi minimum compressive strength.
- For slab on grade, use a 1:3:6 mix ratio (cement: sand:gravel) to achieve 2200 psi minimum compressive strength.
- Cement should be CEM-II, BM Portland Composite Cement.
- Only use clean river sand in the concrete mix.
- Never use white quarry sand or beach sand.
- Use clean water in the concrete mix and do not use too much water.
- Use crushed gravel rather than rounded river stones as the coarse aggregate.
- Do not use gravel greater than 20 mm in dimension.
- Ensure that the concrete mix is consolidated and distributed around reinforcement with no voids.

2.16.2.2.3 Brick work

Mortar was mixed at 1:3 in the same manner as concrete. A mix of 1:2 was used for the damp proof course and the walls in the bathroom. Because the bricks are so porous, they have a tendency to absorb water from the mortar before the cement has time to hydrate and create a strong bond. We promoted wetting or soaking the bricks prior to building the wall. In addition, we stressed uniform joint thickness no greater than 15mm, filling the joints completely with mortar, staggering the vertical joints, and ensuring the wall remained plumb.



a) Good Practice b) Bad Practice

Figure 2.34: Bonding practices of Brick wall

- Use only clean river sand in the mortar mix.
- Never use white quarry sand or beach sand
- Mix dry components until color is uniform prior to adding water.
- Use clean water in mortar mix.
- Add only enough water for workability.
- Mix mortar for 3 minutes
- Use mortar within 1 hour.
- Do not remix mortar.

CHAPTER 3
NUMERICAL MODELING

3.1 General

This chapter describes the numerical Modeling of a low rise building in Conventional and Confined method. A 3-storied RC Frame structured building including foundation in both methods was modeled in 3D finite element software ETABS. Based on the results of finite element structural analysis of ETABS, the building was designed considering conventional foundation system and proposed confined foundation system as per BNBC (1993). After structural analysis and design, materials quantity and bill of quantity was estimated for both the methods, Cost statement and cost comparison was performed in the category of foundation and superstructure separately for both the system.

3.2 Modeling of 3-storied RC frame structured building

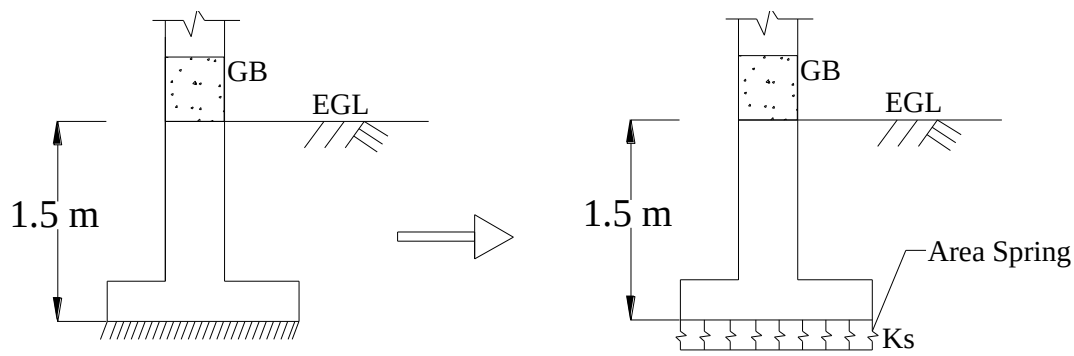
Modeling of a low rise 3-Storied Reinforced concrete (RC) frame structured building including foundation was performed in the methods (1) Conventional method (2) Confined method.

3.2.1. Conventional Method

A typical model of 3-Storied RC frame structured building located in Seismic Zone-II of Bangladesh was prepared in 3D finite element software ETABS considering gravity loads (Dead load and Live load) and earthquake loads with three bay of 3.66 m C/C in XX direction and two bay of 4.57 m C/C in YY direction. The finite element analysis of this model includes modeling of RC columns and beams, slabs, stairs, generation of finite element mesh. RC Columns and Beams were modeled as Line element, RC slabs and stair were modeled by area elements as shell. Mesh size of 4x4 square was adopted for infill wall and slabs; mesh size of 4 was for beams and columns to maintain coincidence of node points of slab and infill wall. Conventionally for frame structure analysis it has been generally found that masonry wall is modeled as load on beam. So in this method masonry walls was considered as non structural elements and loads on beams and slabs as uniformly distributed load.

Dead loads and live loads were calculated as per Bangladesh National Building Code (BNBC-1993). Earthquake loads was applied to the structure as per BNBC (1993) for seismic design category C in moderate seismic risk Zone-II where requirements for frames to make the frame earthquake resistant as per section 8.3.10, BNBC (1993) was ensured as Intermediate Moment Resisting Frame.

Foundation of conventional method was considered as isolated column footing, conventionally designed in rural and urban areas of Bangladesh and modeled as hinge in preliminary ETABS model. Foundation model has shown in figure 3.1.



Conventional Foundation Model

Figure 3.1: Conventional foundation model

Table 3.1: Site investigation data/foundation data

Data Name	Symbol	Unit	Value
Allowable bearing capacity of soil	q_a	kPa	As per Table 4.4
Factor of safety	FS	-	3.0
Allowable settlement	ΔH	mm	50
Foundation depth	D	m	1.5
Un-drained cohesion (for stiff clay)	C_u	kPa	50
Compression index	C_c	-	0.1
Initial void ratio	e_o	-	0.75

Considering sub-soil of Holocene age, above foundation data was taken. Also according to BNBC (1993), Table 6.3.5, Part-6, the presumptive value of bearing capacity is 100 kPa for uniform soft clay that can be indented with modest thumb pressure in case of lightly loaded structures if soil investigation data is unavailable.

For determining bearing capacity of foundation soil both bearing capacity analysis and settlement analysis was carried on and a 5.0 meter clay soil layer was considered beneath the footing which e_o=0.75 and C_c=0.1. Foundation of conventional method was modeled by hinge in preliminary model to determine tentative footing size. After that settlement analysis was carried on to determine spring constant or modulus of subgrade reaction (K_s) and thus hinge support was replaced by Area spring. After putting the tentative footing size in foundation with area spring, run the model for analysis and found axial load varies with preliminary model. By taking the axial load from trail 1 analysis, again settlement analysis was carried on and re-calculated the footing size and Area spring constant. Tolerance of axial load was taken 5% to 10% as limit and as thus footing size, Area spring constant finalized by trial and error method. Modulus of subgrade reaction (K_s) was calculated for a factor of safety 3.0 (Bowles, 1997) and applied in Local 3 direction in ETABS model. Allowable bearing capacity was taken as minimum of both settlement analysis and bearing capacity analysis. Foundation design was performed by this allowable bearing capacity.

Table 3.2: Material Properties of Concrete

Property Name	Unit	Value
Material type	-	Isotropic
Weight	kN/m^3	23.50
Modulus of Elasticity, E_C	MPa	15451
Poisson's ratio, ν	-	0.17
Coefficient of thermal expansion, α_t	$10^{-6}/K$	12
Shear Modulus, G	MPa	6603
Compressive strength of concrete at 28 days, f'_c	MPa	17.5
Yield stress of reinforcement, f_y	MPa	414

Table 3.3: Section Properties of Frame Members

Member	Dimension
Floor Beam (width x height)	254 mm X 381 mm
Grade Beam (width x height)	305 mm X 305 mm
Column C1	305 mm X 305 mm
Column C2, C3 and C4	305 mm X 381 mm
Slab	Thickness=100 mm
Stair	Thickness=152 mm

Table 3.4: Story Height Data

Story	Height
Typical Storey	3.05 m
Chilecota	2.44 m

Table 3.5: Static Load Cases and Loads

Load Name	Load Type	Details	Value
DEAD	Dead Load	Self Weight of Structural Members was calculated automatically using self weight Multiplier in ETABS	-
		Uniform Dead Load on Floor Slab	0.96 kN/m^2
		Uniform Dead Load on Roof Slab (RT)	1.44 kN/m^2
LIVE	Live Load	Uniform Live Load on Floor Slab	2.0 kN/m^2
		Uniform Live Load on Roof Slab	1.5 kN/m^2
		Uniform Live Load on Stair Case	4.0 kN/m^2

Table 3.6: Static Earthquake Load Parameters (BNBC-1993)

Parameter	Value	Remarks
Seismic Zone Factor, Z	0.15	Moderate Seismic Zone-II
Site Co-efficient, S	1.5	Table 6.2.25, Part 6 in BNBC (1993)
Overstrength Factor, R	8.0	Table 6.2.24 in BNBC (1993) Intermediate Moment Resisting Frame
Importance Factor, I	1.0	Table 6.2.23 in BNBC (1993)

C_t	0.073	Section 2.5.6.2 in BNBC (1993)
Fundamental Period, T	0.45 sec	Section 2.5.6.2 in BNBC (1993)
Seismic Design Category (SDC)	C	Structure in Seismic Zone-II as per BNBC (1993).
Direction of Loading	X and Y	-

3.2.2. Confined Method

An infill wall constructed after construction of RC frames and an infill wall constructed as an integrated part of RC frames will behave differently. This effect of construction sequence will be considered in the proposed new foundation system. In the proposed foundation system, RC wall footing will be constructed first; infill masonry wall will be constructed on RC wall footing keeping space for columns. When masonry wall will reach up to the bottom of the grade beam, the columns will be constructed. Then grade beams will be constructed directly on masonry infill wall and columns.

By considering this integrated or composite structure, a typical model of 3-storied RC frame structured building with confined infill brickwall located in seismic zone-II of Bangladesh was prepared in 3D finite element software ETABS considering gravity loads (Dead load and Live load) and earthquake loads with three bay of 3.66 m C/C in XX direction and two bay of 4.57 m C/C in YY direction. In confined method, finite element model was constructed by direct frame elements commonly known as confining elements and masonry wall reported as continuum idealization for the infill panel and frame (Taher and Afefy, 2008) i.e. finite element method as it can provide an accurate computational representation of both material and geometry aspects (Korkmaz, Demir Sivri, 2007). The infill panel is macro mechanically treated as homogeneous material and the effect of mortar joints between masonry units is smeared over the whole panel and taken on an average sense (Haddad, 1991) and so

on an average compressive strength & modulus of elasticity was taken for analysis. The finite element analysis of this model includes modeling of RC columns and beams, Masonry Walls, slabs, stairs, generation of finite element mesh. RC columns and beams were modeled as Line element, RC slabs and stair were modeled as area elements, and masonry wall was also modeled as area elements with opening for windows and doors. Confined infill walls were modeled as 4 noded plane stress elements. In this method masonry walls were considered as structural elements i.e. load bearing wall and RC lintel was provided above all opening having 300 mm bearing. Since the brick masonry walls have only compressive strength and stiffness, i.e. they cannot resist bending in any direction, bending stiffness of masonry wall was made as near to zero in ETABS model. Extra RC confining columns was provided in both sides of doors as opening area is greater than 1.5 m^2 as per eurocode8.

The gap element which is normally used to model the contact between two structures was used as the interface elements connecting the infill wall to surrounding frame considering the friction coefficient. Gap elements have the capability to transfer the loads from frame to infill walls thus allowing the influence of infill on the frame structure to be modeled.

In the absence of definite information on the friction coefficient, μ Doudoumis and Mitsopoulou (2007) used two separate analyses, one with μ=0 and other with μ=1 for a model with contact surface. Accordingly a static analysis was performed to study the stiffness of gap element required to produce the same roof displacement. The same procedure was repeated for different strength of infill materials ranging expressed in terms of young's modulus, (E_i) from 1 GPa to 15 GPa while other parameter such as horizontal load, material property, cross section of concrete members and the thickness of the infill were kept the same and the results are shown in Table 3.7.

Table 3.7: Gap Stiffness in N/mm as per Doudoumis and Mitsopoulou (2007)

Infill stiffness, K_i (=E_it) 10^6 (N/mm)	Gap stiffness (K_g) Corresponding to μ=0 (N/mm)	Gap stiffness (K_g) Corresponding to μ=1 (N/mm)
0.2	1600	4500
0.5	12500	23000
1.0	30000	50000
1.5	50000	70000
2.0	63000	107000
2.5	70000	120000
3.0	82000	130000

The gap stiffness shown in table 3.7 is for two extreme cases; frictionless contact (μ=0) and 100% frictional contact (μ=1) at the interface. These extreme cases may not exist in physical conditions due to the presence of mortar at the interface between the frame and the infill.

Dorji (2009) drawn the figure 3.2 which shows the average gap stiffness required to simulate different strengths of infill material approximately corresponding to a friction coefficient μ=0.5. According to BNBC (1993), section 6.13.3.15, coefficient of friction is 0.6 for concrete placed against hardened concrete not intentionally roughened. From web search it was found that, for brick masonry on dry clay, μ=0.5. But standard coefficient of friction between masonry wall and concrete was not found in any code or any literature. From the literature review it was learnt that contact friction at interface cannot be quantified as it depends on many factors such as quality of materials, quality of workmanship (Moghaddam, 1987). So a coefficient of friction, μ=0.5 was taken for gap or interface elements stiffness calculation. Thus we can follow the procedure and equation of Dorji (2009) for gap element stiffness.

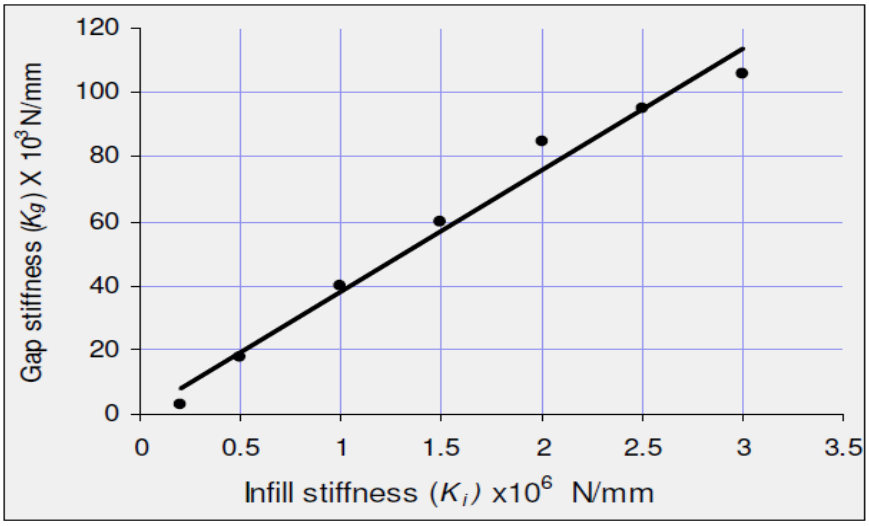


Figure 3.2: The Gap stiffness (K_g) and infill stiffness (K_i), Dorji (2009)

Infilled structures consist of low strength mortar at the frame infill interface. To simulate such a condition, an effective stiffness of the gap elements can be calculated by the relation 3.1 (Dorji, 2009). The effective stiffness property of the gap element was investigated by Dorji (2009) through a trial and error procedure to match the results of previous research of Doudoumis, (2007). The linear relationship between the gap and infill stiffness as shown in figure 3.2 could be used in modeling the interface element between the frame and infill for capturing the global structural response of a confined infilled structures. Dorji (2009) proposed the following equation to calculate the gap elements stiffness:

K_g=0.0378 K_i+347……..3.1

Where,

K_g=effective stiffness of gap element in N/mm

K_i=E_it =effective stiffness of infill panel in N/mm

E_i=modulus of elasticity of infill panel in MPa

t= thickness of infill panel in mm

Dorji (2009) repeated the analysis once again using the gap stiffness of equation 3.1 and prove that confined infilled frame system have higher stiffness than the conventional bare frame system under lateral load by comparing the results of load

deflection curve. Dorji (2009) also calculated the roof displacement of the model by using equation 3.1 and compared with the values of Doudoumis (2007). Dorji (2009) validated his equation by proving that his strength-displacement curve as shown in figure 3.3 (Current model indicated μ=0.5) lies between the curves proposed by Doudoumis (2007) for μ=0 and 1.

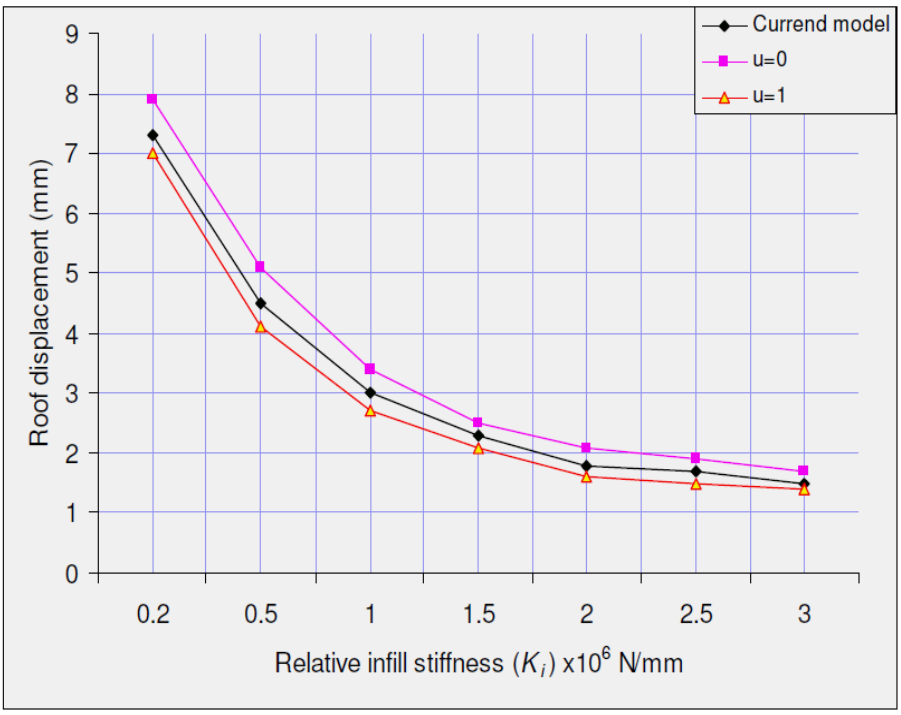


Figure 3.3: Validation of the Model Proposed by Dorji (2009)

From equation 3.1, effective stiffness of gap element was found 42,021.5 N/mm and effective damping of gap element of 3% was used as per recommendation of Chopra, (2000).

The effect of infill panel is significantly depends on the size of finite element mesh. The infill wall tends to behave as shear panel when mesh size is large and finer mesh increases the accuracy of the results (Doudoumis, 2007). Dorji (2009) suggested ratio of mesh size to panel size should be less or equal to 0.08 to avoid the infill panel behave as shear panel. However, (Doudoumis, 2007) shows that, it becomes evident that a mesh density of 4 × 4 gives satisfactory results for the total stiffness of the system and it is quite satisfactory for the stress distribution within the infill panel. So

combining all suggestions, a mesh size of 4x4 squares was adopted for infill wall and slabs; mesh size of 4 was for beams and columns to maintain coincidence of node points of slab and infill wall.

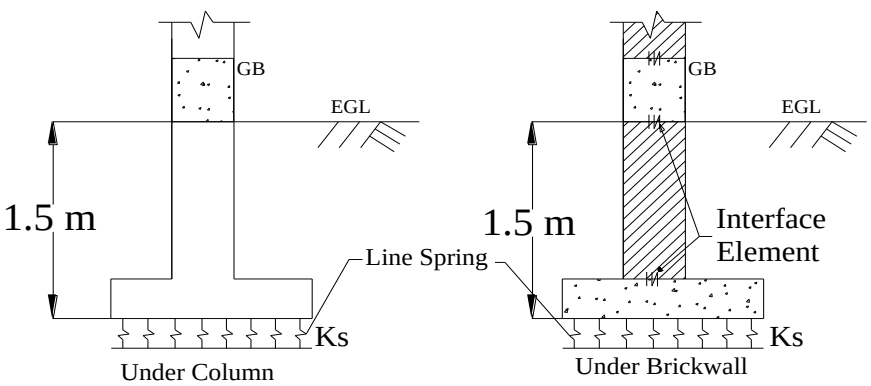
Dead loads and live loads were calculated as per Bangladesh National Building Code (BNBC-1993). Earthquake loads was applied to the structure as per BNBC (1993) for Seismic Design category C in moderate seismic risk Zone-II where requirements for frames to make the frame earthquake resistant as per 8.3.10, BNBC (1993) has ensured as Intermediate Moment Resisting Frame.

Considering foundation system is integrated with frame structure (as a whole rigid body) in the Confined method. Foundation of confined method was modeled by line spring under a beam where modulus of subgrade reaction (K_s) was calculated for a factor of safety 3.0 (Bowles, 1997) and applied in Local 2 direction in ETABS model. It has 250 mm load bearing Masonry wall with foundation all along the periphery of the building from footing to grade beam bottom. Finite element auto mesh was also done for this load bearing wall. Gap element was used as interface elements between RCC base, Grade beam and confined masonry wall in foundation. Figure 3.4 shows the foundation model of the confined method.

According to Bowles (1997), Modulus of Subgrade Reaction K_s used for structural analysis of foundation $= \frac{Q\ ult}{\Delta H}$ ……. (3.2)

Where, Q_{ult}=Ultimate Bearing capacity of foundation soil

ΔH=Total Foundation Settlement



Confined Foundation Model

Figure 3.4: Foundation model of confined method

Table 3.8: Site investigation data/foundation data

Data Name	Symbol	Unit	Value
Allowable bearing capacity of soil	q_a	kPa	As per Table 4.5
Factor of safety	FS	-	3.0
Allowable settlement	ΔH	mm	50
Foundation depth	D	m	1.5
Un-drained cohesion (for stiff clay)	C_u	kPa	50
Compression index	C_c	-	0.10
Initial void ratio	e_o	-	0.75

Considering sub-soil of Holocene age, above foundation data was taken. Also according to BNBC (1993), Table 6.3.5, Part-6, the presumptive value of bearing capacity is 100 kPa for uniform soft clay that can be indented with modest thumb pressure in case of lightly loaded structures if soil investigation data is unavailable.

For determining bearing capacity of foundation soil both bearing capacity analysis and settlement analysis was carried on and a 5.0 meter clay soil layer was considered beneath the footing which e_o=0.75 and C_c=0.1. Foundation of confined method was modeled by line spring under a beam in preliminary model to determine tentative footing size. After that settlement analysis was carried on to determine spring constant or modulus of subgrade reaction (K_s). After putting the tentative footing size in foundation with line spring, run the model for analysis and found axial load varies with preliminary model. By taking the axial load from trail 1 analysis, again settlement analysis was carried on and re-calculated the footing size and line spring constant. Tolerance of axial load was taken 5% to 10% as limit and as thus footing size, line spring constant finalized by trial and error method. Allowable bearing capacity was taken as minimum of both settlement analysis and bearing capacity analysis. Foundation design was performed by this allowable bearing capacity.

.Table 3.9: Material Properties of Concrete

Property Name	Unit	Value
Material type	-	Isotropic
Weight	kN/m^3	23.50
Modulus of Elasticity, E_C	MPa	15451
Poisson's ratio, ν	-	0.17
Coefficient of thermal expansion, α_t	$10^{-6}/K$	12
Shear Modulus, G	MPa	6603
Compressive strength of concrete at 28 days, f'_c	MPa	17.5
Yield stress of reinforcement, f_y	MPa	414

Table 3.10: Material Properties of Confined infill brick wall

Property Name	Unit	Value
Material type	-	Isotropic
Weight	kN/m^3	18.9
Modulus of Elasticity, E_m or E_i	MPa	7875
Poisson's ratio, ν	-	0.15
Coefficient of thermal expansion, α_t	$10^{-6}/K$	8.0
Shear Modulus, G	MPa	7756
Compressive strength of masonry wall, f'_m	MPa	10.5

Modulus of Elasticity of Masonry Wall (E_m) was taken equal to 750 f'_m as per BNBC (1993), section 4.3.8. Compressive Strength of Masonry Wall, f'_m was taken 10.5 MPa with Grade of Mortar M1 (Proportion 1:3 of cement: sand) and first class burnt clay bricks locally available as per BNBC (1993), Table 6.4.1.

Table 3.11: Section Properties of Frame Member and Infill Wall

Member	Dimension
Floor Beam (width x height)	203 mm X 178 mm
Grade Beam (width x height)	254 mm X 203 mm

Column C11	203 mm X 203 mm
Column C12	228 mm X 203 mm
Column C13	152 mm X 152 mm
Slab	Thickness=100 mm
Stair	Thickness=152 mm
Infill Masonry Wall	Thickness=125 mm

Table 3.12: Story Height Data

Story	Height
Typical Storey	3.05 m
Chilecota	2.44 m

Table 3.13: Static Load Cases and Loads

Load Name	Load Type	Details	Value
DEAD	Dead Load	Self Weight of Structural Members was calculated automatically using self weight Multiplier in ETABS	-
		Uniform Load on Floor Slab	0.96 kN/m^2
		Uniform Load on Roof Slab (RT)	1.44 kN/m^2
LIVE	Live Load	Uniform Load on Floor Slab	2.0 kN/m^2
		Uniform Load on Roof Slab	1.5 kN/m^2
		Uniform Load on Stair Case	4.0 kN/m^2

Table 3.14: Static Earthquake Load Parameters (BNBC-1993)

Parameter	Value	Remarks
Seismic Zone Factor, Z	0.15	Moderate Seismic Zone-II
Site Co-efficient, S	1.5	Table 6.2.25, Part 6 in BNBC (1993)
Overstrength Factor, R	7.0	Table 6.2.24 in BNBC (1993) Dual system.
Importance Factor, I	1.0	Table 6.2.23 in BNBC (1993)

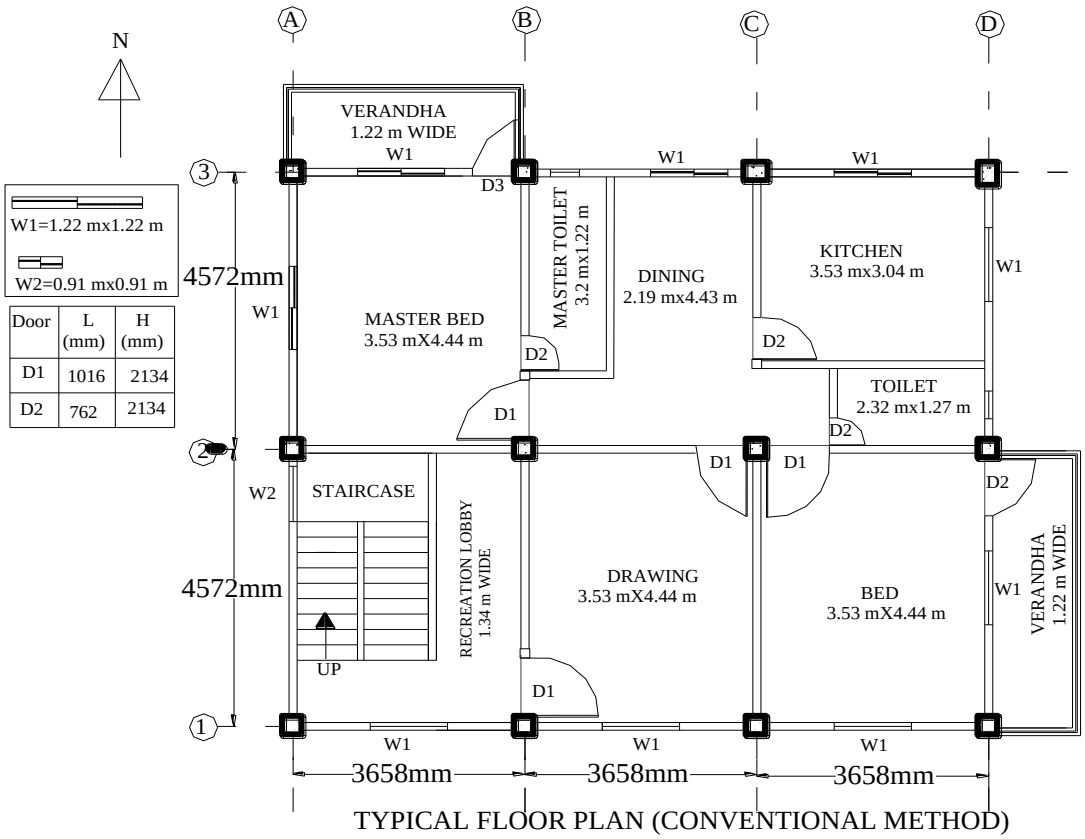
C_t In XX direction In YY direction	0.0275 0.0236	Section 2.5.6.2 in BNBC (1993)
Fundamental Period, T In XX direction In YY direction	0.169 sec 0.145 sec	Section 2.5.6.2 in BNBC (1993)
Seismic Design Category (SDC)	C	Structure in Seismic Zone-II as per BNBC (1993).
Direction of Loading	X and Y	-

3.3 Structural Analysis and Design of Foundation and Super-structure

Finite Element Technique is a powerful and versatile tool for the analysis of problems of structural and continuum mechanics. In this projects works to determine the response of the structure a linear finite element static analysis has been performed using the package ETABS to predict the inelastic behavior of RC frame and infill masonry wall.

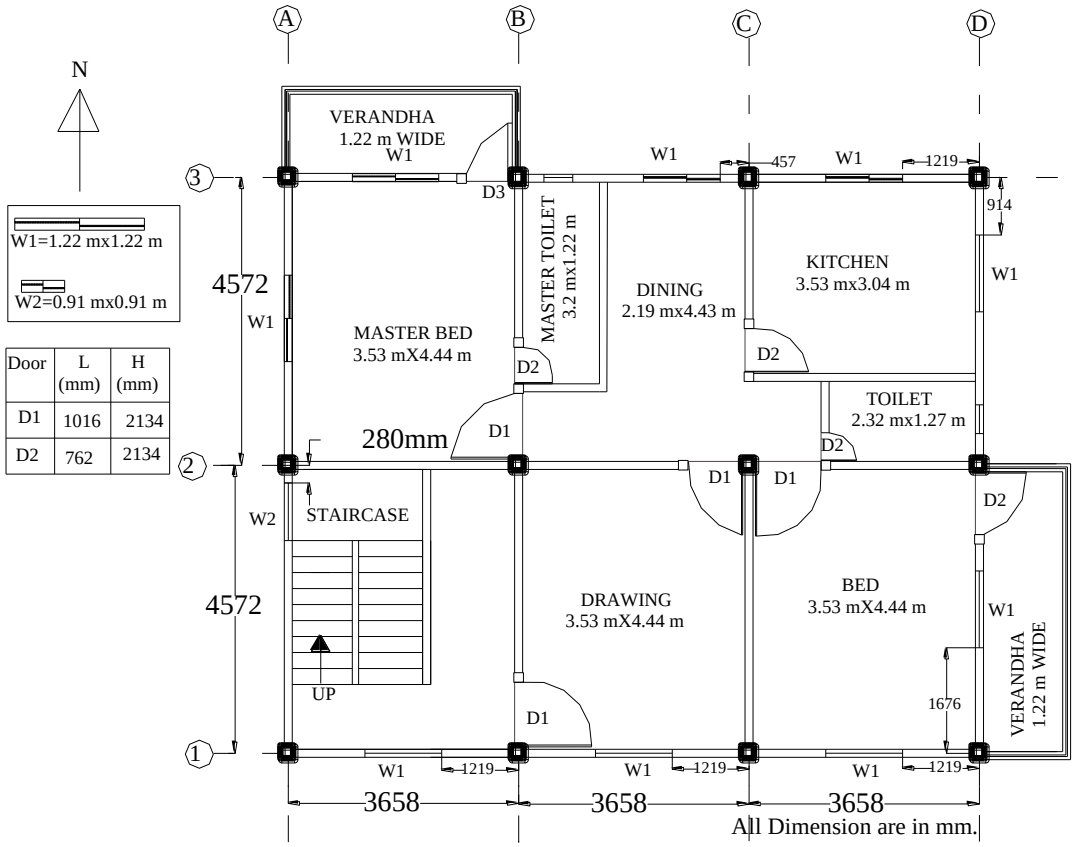
In conventional method, isolated column footing was considered as load transfer medium of the structure to the soil. Foundation of conventional system was designed manually by taking axial force, Moments from the ETABS finite element analysis. In Confined method, continuous masonry wall footing with RC base was considered as load transfer medium of the structure to the soil where soil-structure interaction was considered and modeled by line spring on beam. Foundation of Confined method was designed manually by taking spring forces from the ETABS analysis. BNBC (1993) was followed in foundation design. Both the system of Foundation was checked against earthquake loads. Footing size was determined based on allowable loads but footing depth and reinforcement design was based on ultimate strength design (USD) method.

After structural analysis, structural design of frames was workout in ETABS as per the Code BNBC (1993). Frame member like Column and Beam was directly designed in ETABS. Other element like slabs, stair were designed manually as per BNBC (1993). In conventional method, earthquake resistant seismic detailing of the buildings was performed as per BNBC (1993) considering the basic structural system is Intermediate Moment resisting frame where provisions of seismic resistant detailing of frame structure located in Seismic Zone-II are applicable. But in Confined method, earthquake resistant seismic detailing of the buildings was performed as per eurocode8 (2003) due to the absence of seismic design and detailing guideline in BNBC (1993) for confined masonry buildings. In confined method the basic structural system was considered as dual system where lateral force resisting system was masonry shear wall with concrete Intermediate Moment resisting frame. Default load combinations of concrete frame design of BNBC (1993) were considered in structural design. Ultimate strength design (USD) procedure was followed in structural design.



Door	L (mm)	H (mm)
D1	1016	2134
D2	762	2134

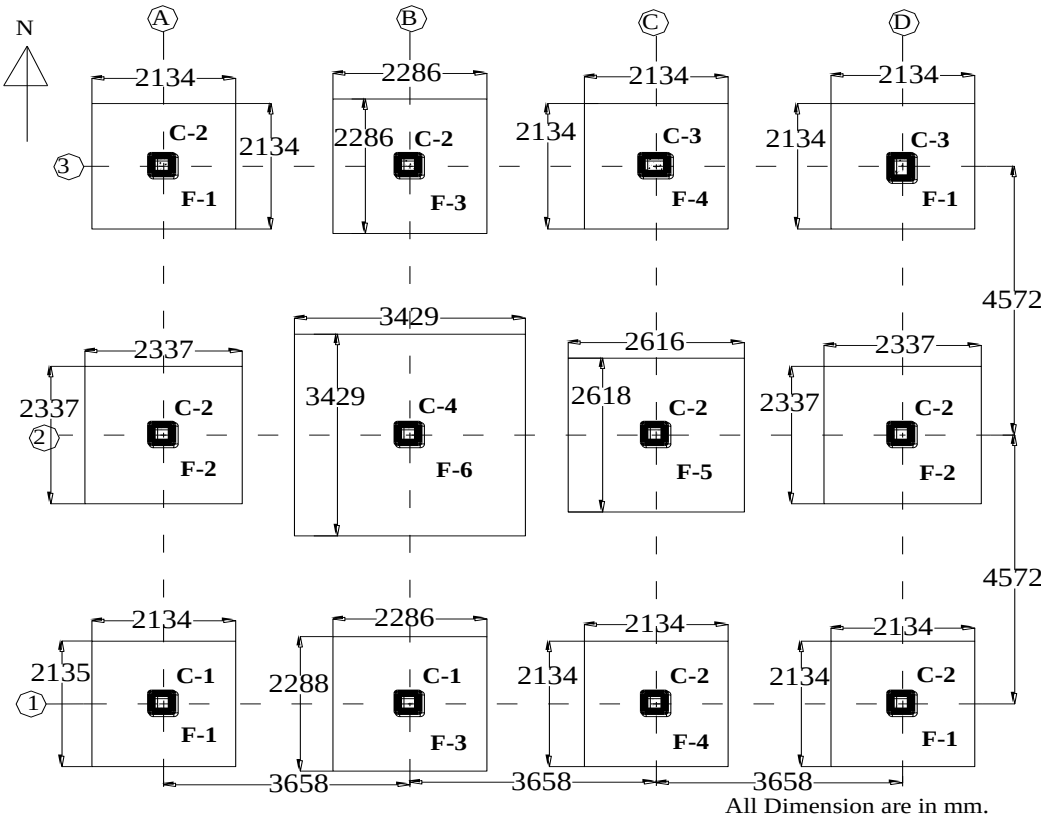
TYPICAL FLOOR PLAN (CONVENTIONAL METHOD)



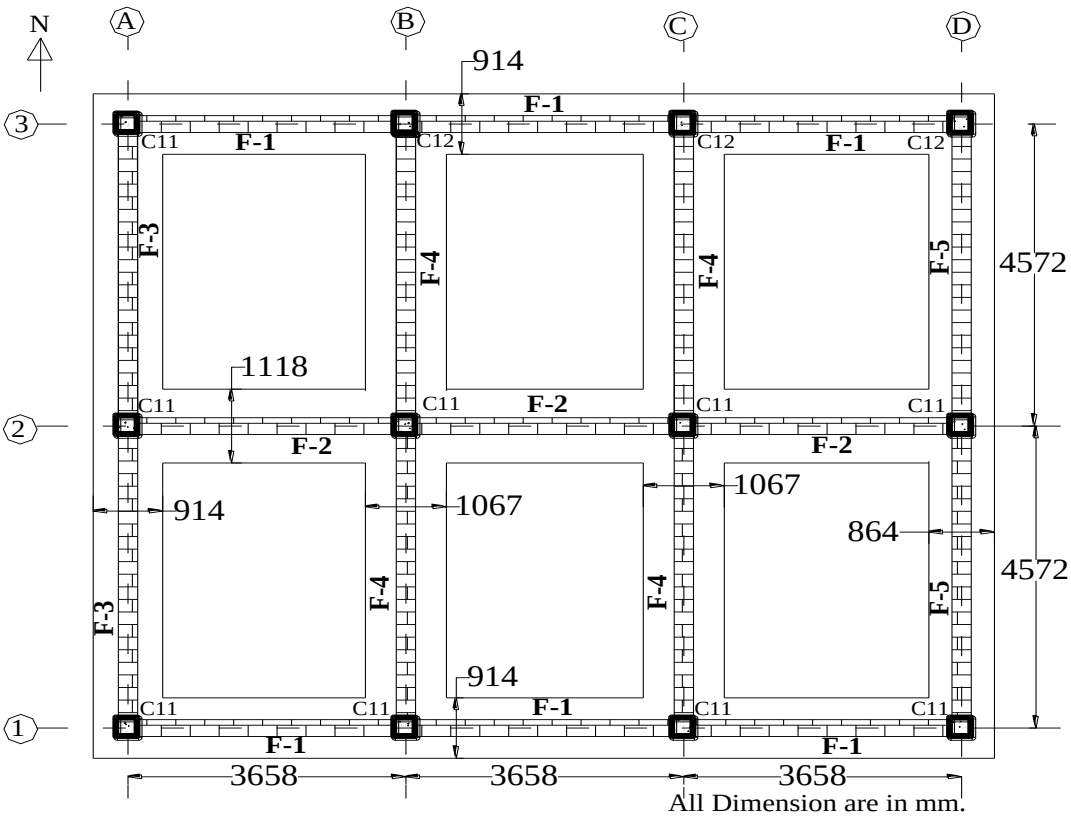
Door	L (mm)	H (mm)
D1	1016	2134
D2	762	2134

TYPICAL FLOOR PLAN (CONFINED METHOD)

Figure 3.5: Typical Floor Plan of Conventional and Confined Method



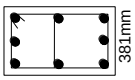
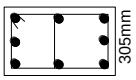
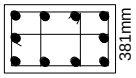
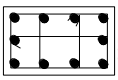
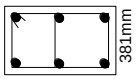
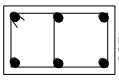
FOOTING LAYOUT PLAN (CONVENTIONAL METHOD)

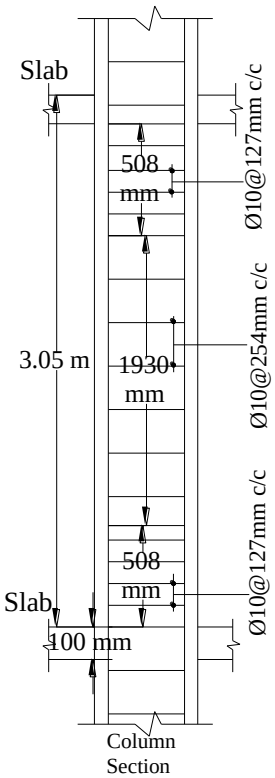


FOOTING LAYOUT PLAN (CONFINED METHOD)

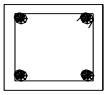
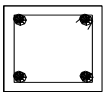
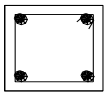
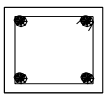
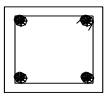
Figure 3.6: Foundation Layout Plan of Conventional and Confined Method

COLUMN SCHEDULE (Conventional Method)

Index	Up to Plinth Level (PL)	PL to Roof	Reinforcement Details
C1	381mm x381mm	305mm x305mm	Main bar: 6-Ø20 mm Tie:Ø10mm127/254/127mmc/c
C2	 457mmx381mm	 381mmx305mm	Main bar: 8-Ø20 mm Tie:Ø10mm127/254/127mmc/c
C3	 457mmx381mm	 381mmx305mm	Main bar: 10-Ø20 mm Tie:Ø10mm127/254/127mmc/c
C4	 457mmx381mm	 381mmx305mm	Main bar: 6-Ø16 mm Tie:Ø10mm127/254/127mmc/c



COLUMN SCHEDULE (Confined Method)

Index	Up to Plinth Level (PL)	PL to Roof
C11	 Size:280mm x 280mm Main:4-Ø12mm Tie: Ø8mm@152mm c/c Clear Cover=75mm	 Size: 203mm x 203mm Main:4-Ø12mm Tie: Ø8mm@152mm c/c Clear Cover=30mm
C12	 Size: 304mm x 280mm Main:4-Ø12mm Tie: Ø8mm@152mm c/c Clear Cover=75mm	 Size: 228mm x 203mm Main:4-Ø12mm Tie: Ø8mm@152mm c/c Clear Cover=30mm
C13	Confining column	 Size: 152mm x 152mm Main:4-Ø8mm Tie: Ø8mm@152mm c/c Clear Cover=30mm

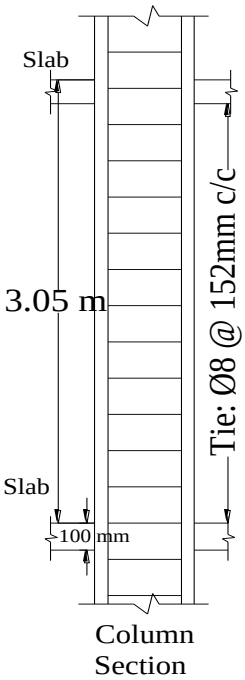
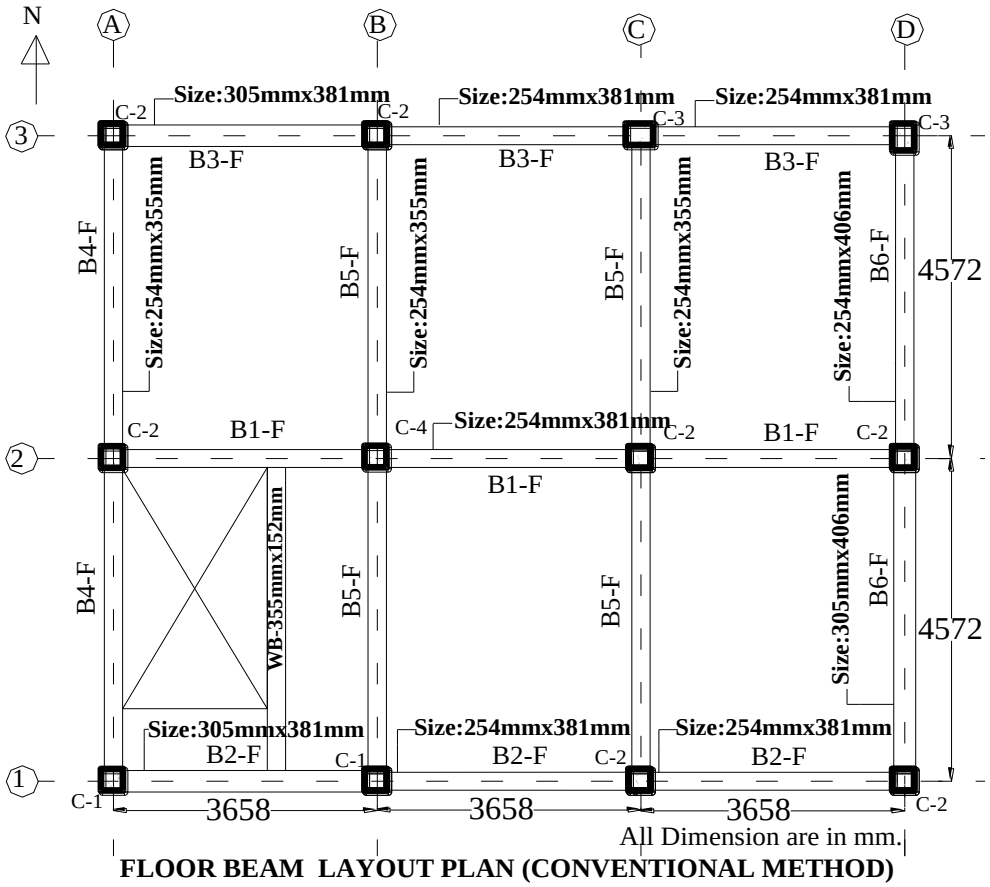
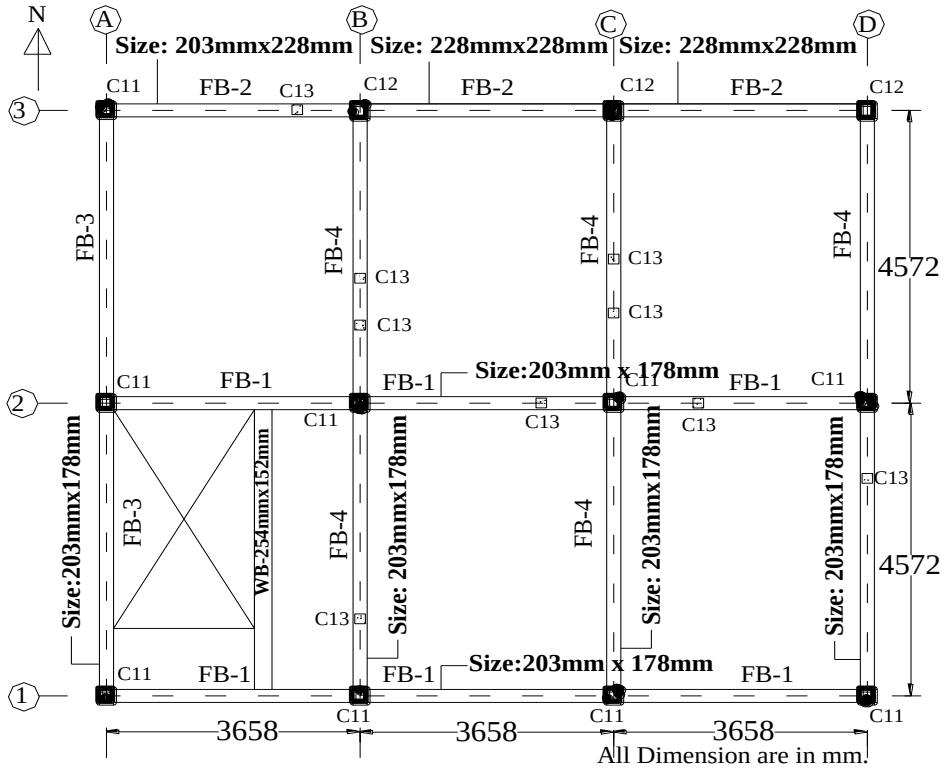


Figure 3.7: Column Schedule of Conventional and Confined Method



FLOOR BEAM LAYOUT PLAN (CONVENTIONAL METHOD)



FLOOR BEAM LAYOUT PLAN (CONFINED METHOD)

Figure 3.8: Floor Beam Plan of Conventional and Confined Method`

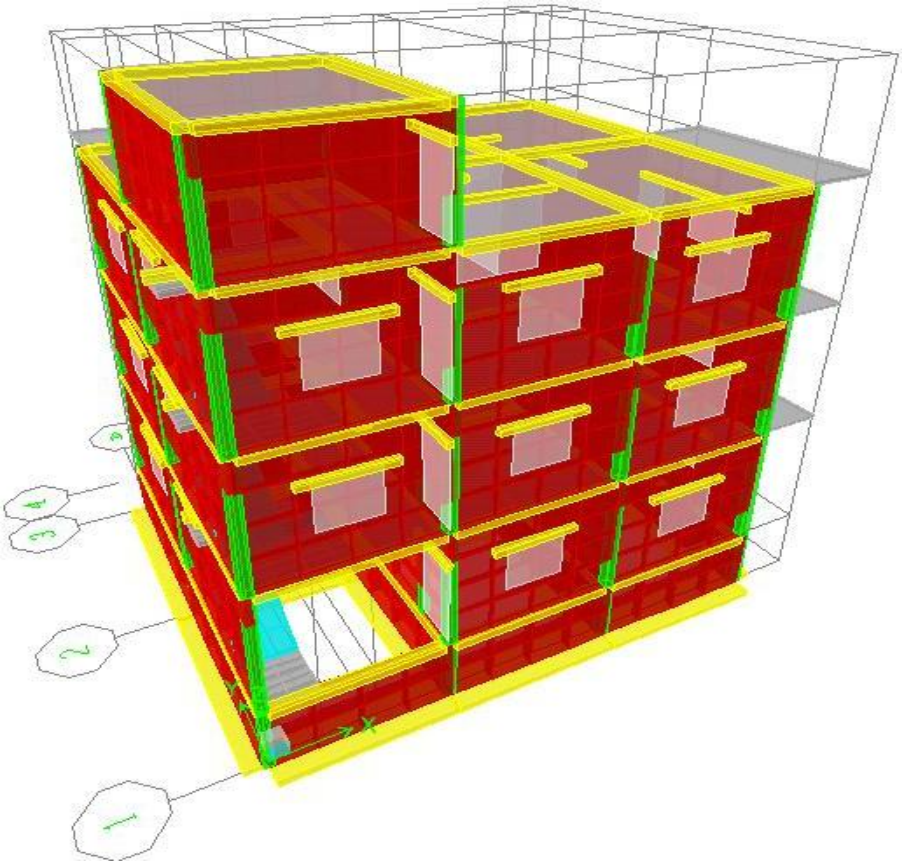


Figure 3.9: 3D Model of Confined Method

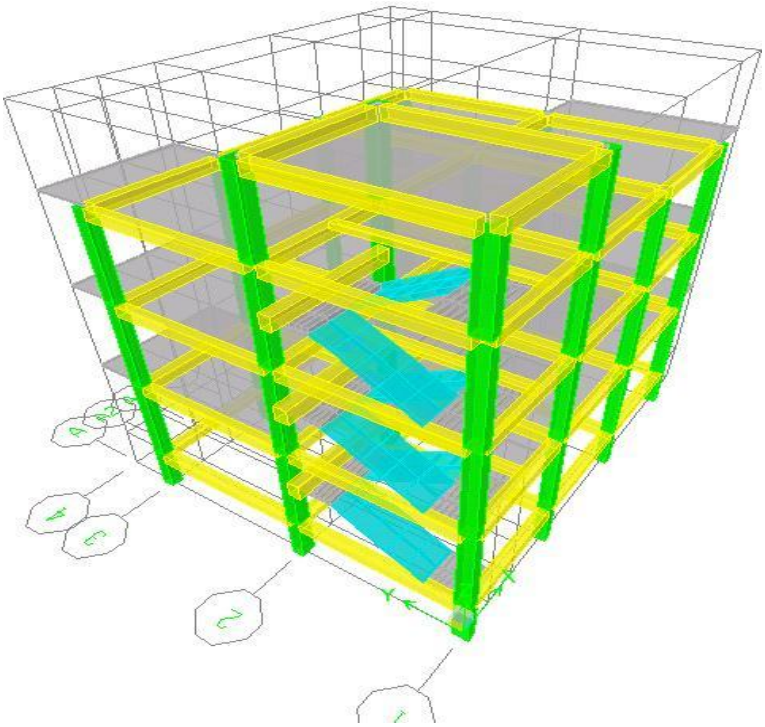


Figure 3.10: 3D Model of Conventional Method

3.4 Optimization of Design

Table 3.15: Interior Column (Conventional Method)

Size of Column (mm)	Moment (kN-m)	Axial Load (kN)	Steel Area (mm^2)	% of Rebar	Cost of concrete & rebar for 3.05 m sample length	Remarks
305x457	47.84	754.8	1394	1.00%	4,910 TK	Un-economic design
305x381	40.76	749.18	1238	1.07%	4,220 TK	Most economic design
305x305	32.73	743.00	2271	2.44%	5,489 TK	Un-economic design
254x305	31.60	734.25	4955	6.40%	9,629 TK	Rebar congestion
254x254	25.20	727.60	4955	7.68%	9,388 TK	Rebar congestion
203x203	14.02	708.00	O/S	O/S	-	Over-stressed (O/S) due to slenderness.

Table 3.16: Exterior Column (Conventional Method)

Size of Column (mm)	Moment (kN-m)	Axial Load (kN)	Steel Area (mm^2)	% of Rebar	Cost of concrete & rebar for 3.05 m sample length	Remarks
305x457	49.44	596.58	2065	1.48%	6,018 TK	Un-economic design
305x381	42.76	590.80	2271	1.96%	5,924 TK	Most economic design
305x305	34.88	583.58	2839	3.05%	6,426 TK	Un-economic design
254x305	32.85	578.82	5110	6.60%	9,885 TK	Rebar congestion
254x254	26.62	572.40	O/S	O/S	-	Over-stressed (O/S) due to slenderness.

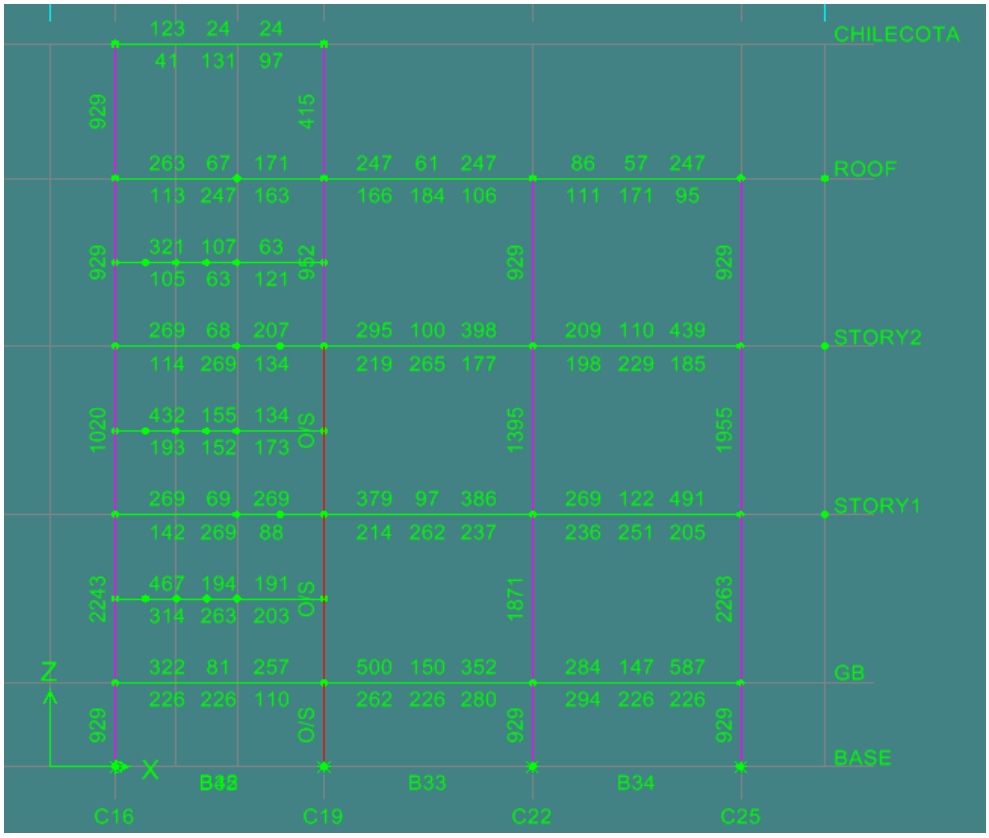


Figure 3.5: Failure of Interior Column in 203x203 mm size (conventional method)

Table 3.17: Interior Beam on grid C (Conventional Method)

Beam size mm	- M kN-m	+ M kN-m	- A_s mm^2	+ A_s mm^2	- A_s %	+ A_s %	Cost of concrete & rebar for 9.15 m length	Remarks
305x381	39.25	43.64	350	392	0.30	0.34	14,143 TK	Costly design
254x355	39.04	36.68	388	362	0.43	0.40	11,785 TK	Economic design
254x254	30.92	22.80	O/S	351	O/S	0.54	-	rebar exceed maximum allowed by code
203x203	19.53	12.75	O/S	284	O/S	0.69	-	rebar exceed maximum allowed

Table 3.18: Exterior Beam on grid A (Conventional Method)

Beam size mm	- M kN-m	+ M kN-m	- A_s mm^2	+ A_s mm^2	- A_s %	+ A_s %	Cost of concrete & rebar for 9.15 m length	Remarks
305x381	31.90	38.21	284	331	0.24	0.29	13,733 TK	Costly design
254x355	24.63	31.16	247	300	0.27	0.33	11,785 TK	Economic design
254x254	21.37	17.75	333	271	0.52	0.42	-	Failed this size due to shear
203x203	30.50	15.53	O/S	220	O/S	0.54	-	Over-stressed (O/S) due to compression failure of beam.

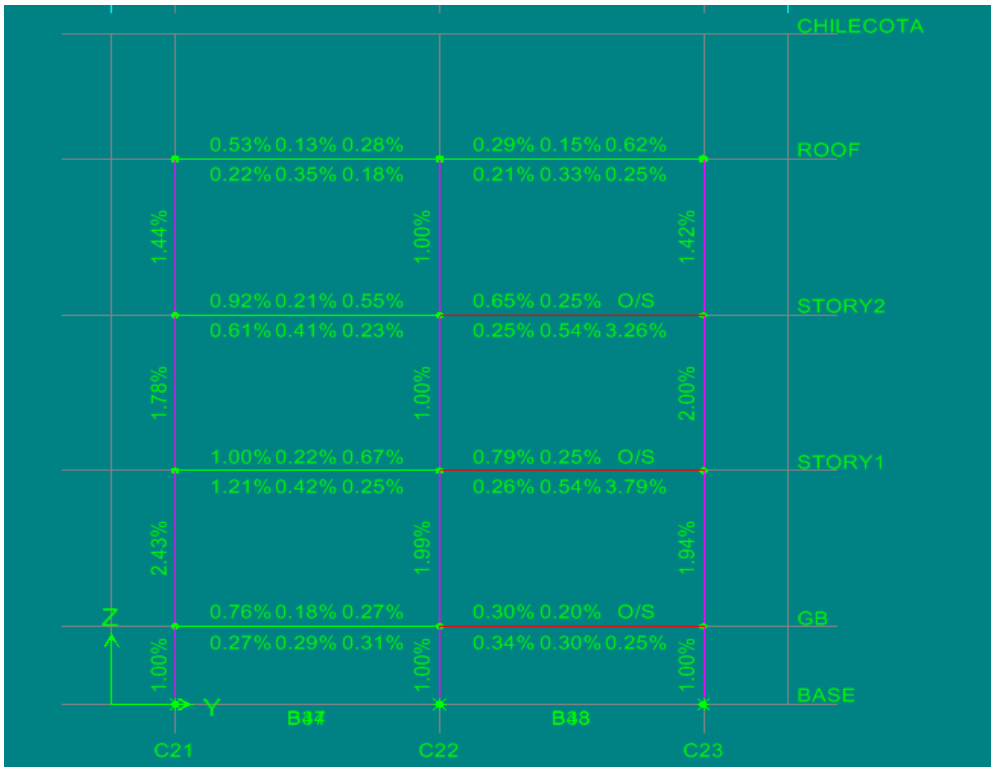


Figure 3.6: Failure of Interior Beam in 254x254 mm size (conventional method)

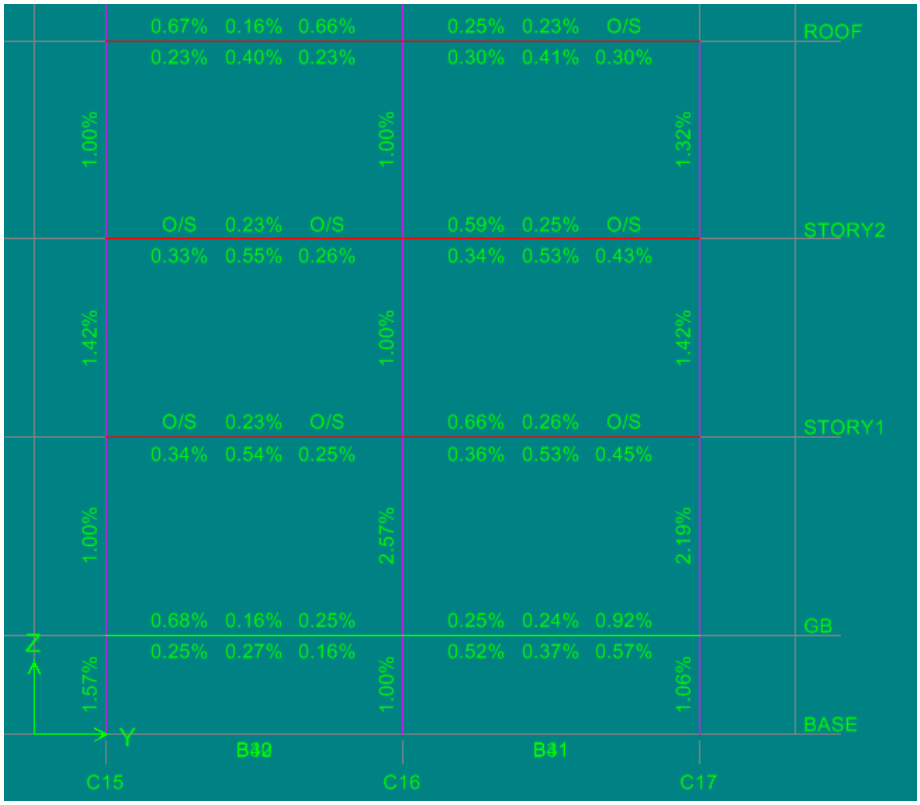


Figure 3.7: Failure of Exterior Beam in 203x203 mm size (conventional method)

3.5 Assessment of Economy of Foundation and Super structure

A cost-benefit analysis was performed for both the foundation system. Quantity of materials and labour was estimated first in the category of

- Civil cost
- Electrical cost
- Plumbing and sanitary cost
- Miscellaneous cost.

Detail bill of quantity (BOQ) was workout then to assess the economy of foundation and super structure of both the system conventional and confined method. Cost comparison table was prepared for the both the foundation systems, civil cost, total cost of the building in aspect of conventional and confined method. BOQ rates were updated rate running in the year of 2012.

CHAPTER 4
RESULTS AND DISCUSSION

4.1 General

The results of the numerical modeling and analytical program has presented in this chapter. After this project work it has been found that, if resistance of masonry infill wall are effectively utilized in bare RC frame structure, it can provide a better, safe and economic solution particularly for low rise buildings of underdeveloped countries, which is subjected to seismic forces and which requires low cost housing at the rural areas that can be capable of making with less skilled worker. Confined method utilized the structural benefits of infill wall in both design and construction and reduced the foundation cost approximately by 21.5% in comparison with conventional bare frame method; also it increases the seismic resistant stiffness of the structure. Comparison of total project cost, seismic resistant capacity of foundation and structure, long term impact and guideline of low cost confined masonry construction also has been presented in this chapter.

4.2 Cost implication

A cost benefit analysis shows that confined method reduced the cost of the project approximately by 15% than the conventionally designed and constructed RC bare frame structure. Cost implication can be broadly discussed in the following sections.

4.2.1 Foundation Cost

A comparison of the foundation cost of proposed confined method and conventional RC bare frame method has prepared. It has been found that confined method reduces the foundation cost approximately by 21.5% in comparisons with conventional bare RC frame method by providing 150 mm thick continuous load bearing masonry wall from footing to grade beam level i.e. as thus designing a continuous wall footing in connection with RC frame member like column, beam etc.

Table 4.1: Foundation Cost comparison of a 3-Storied RC Building

S.l No	Method of design and Construction	Foundation Cost	Cost Variation	Remarks
1	Conventional RC bare frame Method	TK 536,498	21.5%	Confined method cost is less than conventional.
2	Confined Method	TK 421,317		

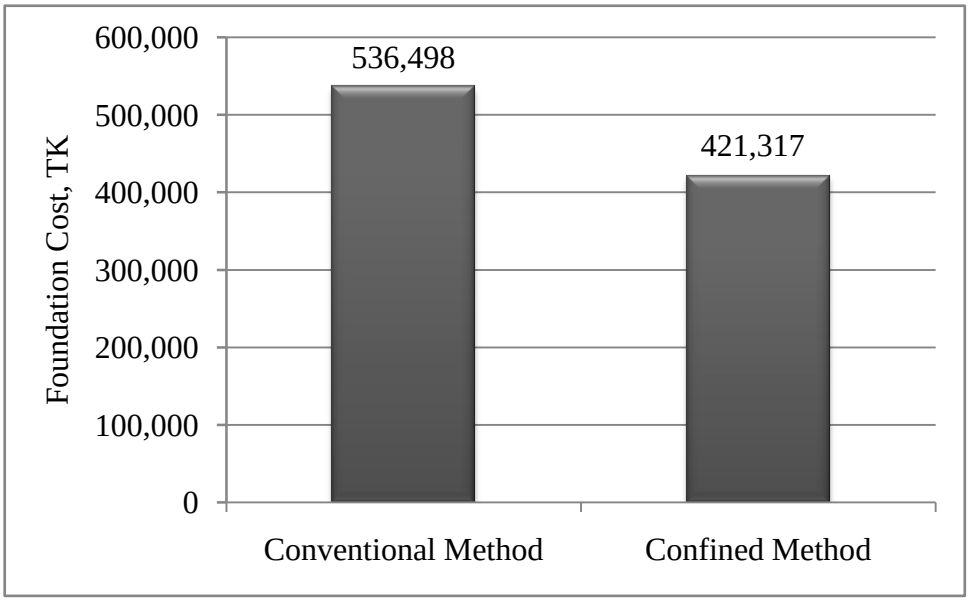


Figure 4.1: Comparison of Foundation Cost of Confined and Conventional Method

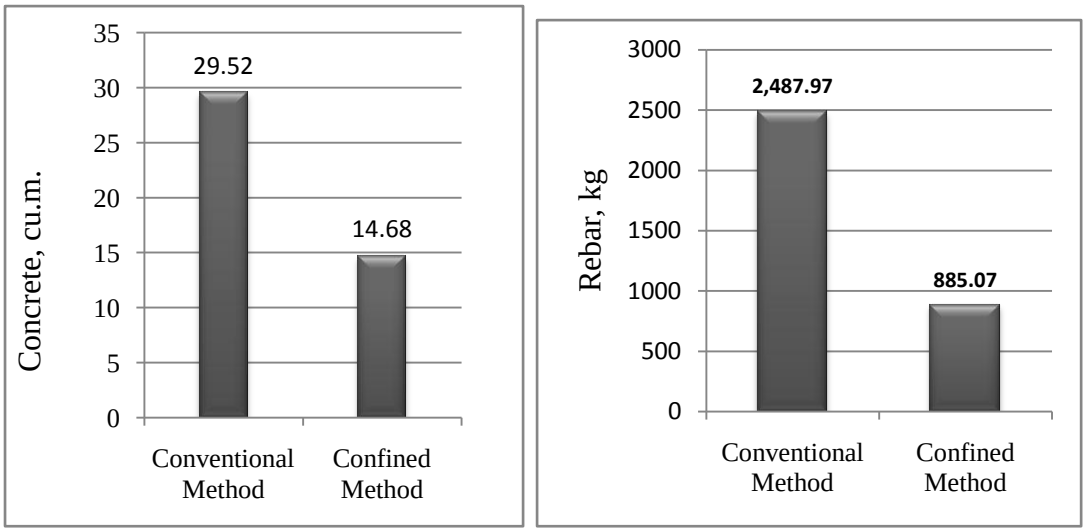


Figure 4.2: Quantity of Concrete and Steel in buildings for two methods (Foundation)

From the above graph (Figure 4.2) it has been clear that, for the same loading, bay, storey concrete volume requited is 50% less and steel volume required is 36% less in the foundation designed in confined masonry construction method rather than conventional RC bare frame method.

4.2.2 Structural Cost of Super-Structure

Detail BOQ has been shows that structural cost of super Structure is 23% less in confined masonry construction method in comparison with conventional RC bare frame method. It included cost of Concrete, Rebar, formwork, brick wall etc.

Table 4.2: Structural Cost of Super Structure comparison of a 3-Storied RC Building

Sl. No	Method of design and Construction	Structural Cost	Cost Variation	Remarks
1	Conventional RC bare frame Method	TK 1,818,672	23%	Confined method cost is less than conventional.
2	Confined Method	TK 1,394,914		

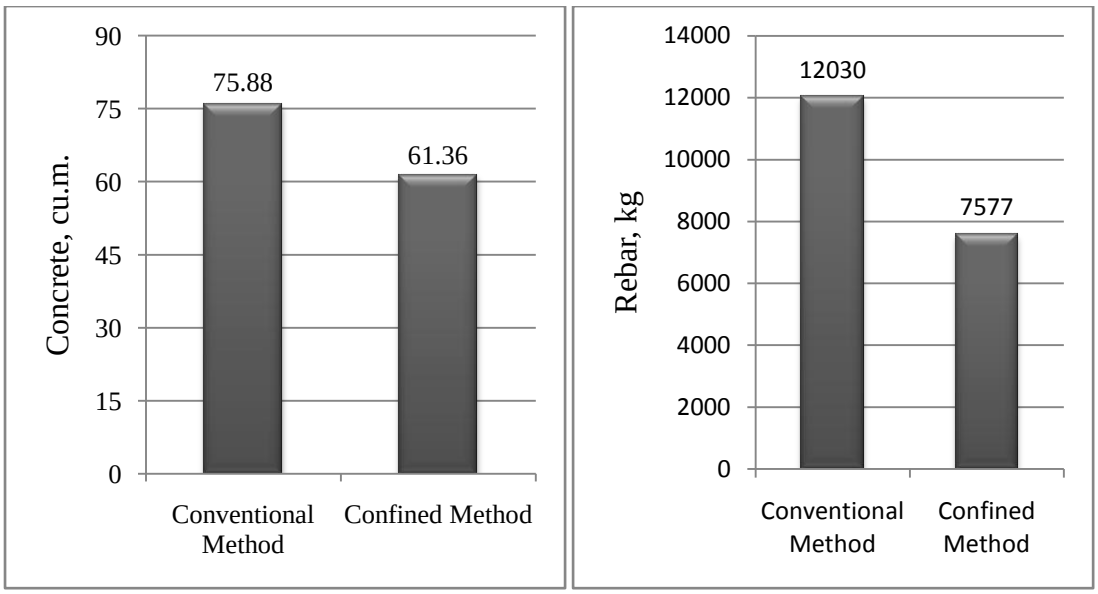


Figure 4.3: Quantity of Concrete and Steel in buildings designed by two methods

From the above graph (Figure 4.4) it has been clear that, for the same loading, bay, storey concrete volume requited in super structure is 19% less and steel volume required is 37% less in confined masonry construction method rather than conventional RC bare frame method.

4.2.3 Total Project Cost of the Building

Total cost of the building includes foundation cost, civil cost, electrical cost, plumbing & sanitary cost and miscellaneous cost. BOQ of both the system shows that total cost of the building is 15% less in confined masonry construction method in comparison with conventional bare frame method. In calculating electrical cost, total electrical material requirement was determined as well as labour cost. This total amount was converted into floor area and stated as floor area basis cost. Plumbing and sanitary cost also calculated in the same procedure.

Table 4.3: Total Project Cost comparison of a 3-Storied RC Building

Sl. No	Method of design and Construction	Foundation Cost	Cost Variation	Remarks
1	Conventional RC bare frame Method	TK 3,934,128	15%	Confined method cost is less than conventional.
2	Confined Method	TK 3,331,894		

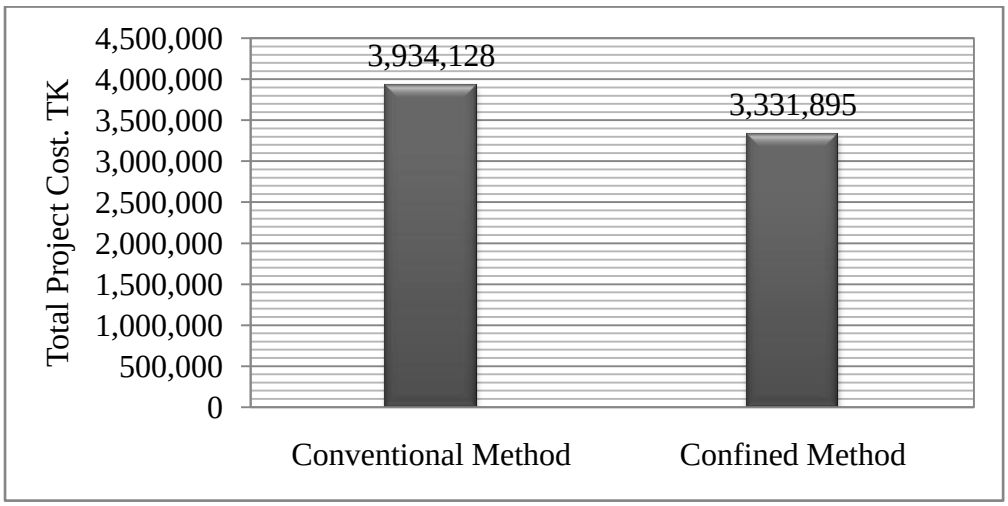


Figure 4.4: Total Project Cost of the building designed by two methods

4.3 Evaluation of Seismic Resistant Capacity

The effect of infill wall predominately changes the behavior of the structure and it is essential to consider infill walls in the analysis and design of building for seismic evaluation of the structure. Finite element analysis shows that the influence of infill on the structural performance is significant. The structural response such as fundamental period, top deflection, inter storey drift ratio, stresses in infill wall and structural member forces of beams and column generally reduce with incorporation of infill wall. Foundation settlement is lower and stiffness of foundation is higher in continuous wall foundation rather than conventional isolated column foundation due to combined action of gravity load and earthquake force. An RC frame structure with masonry infill is more likely to cause deaths and injuries in a strong earthquake than a confined masonry structure. The masonry infill panel in an RC frame structure is vulnerable to collapse in an earthquake due to improper bonding of infill wall and frame.

In a confined masonry structure, the masonry infill can be not only properly restrained from failure, but also leveraged to add strength to the structural system. As thus building designed and constructed in confined masonry method is less vulnerable in earthquake rather than conventional bare frame method. Broader discussion of seismic capacity evaluation has presented in the following sections.

4.3.1 Foundation Settlement and Stiffness

For the purpose of making a building earthquake resistant, it is required to choose an appropriate foundation type for it. Since loads from typical low rise buildings will be light, providing the required bearing area will not usually be a problem. For choosing the type of footing from the earthquake consideration, the soils may be grouped as Firm and Soft avoiding the weak soil unless compacted and brought to Soft or Firm condition. According to IAEE, (2004) those soils which have an allowable bearing capacity of more than 100 kPa is firm soil and with allowable bearing capacity less or equal to 100 kPa is soft soil. In Bangladesh, in most areas top soil layers are soft, usually clay. The allowable bearing capacity considered in the foundation design of both the methods is in the range of soft soil. Continuous masonry wall footings with

RCC base are considered to be most effective from earthquake considerations in firm and soft soil to resist the shear failure and sleep of soft soil beneath the footing (IAEE, 2004). But excessive settlement occurs beneath individual column footing in soft soil when shear failure and sleep of the soil has occurred. The depth of footing has kept well below the lowest level of weathering (at 1.5 m level) as per BNBC (1993).

A serious problem of earthquake is subsidence of the soil beneath foundation. During earth ground shaking, compaction of the soil particles occurs because the individual soil grains are rearranged to take up less space. When the volume of soil is reduced in this manner, the soil beneath foundation settles, forming a depression which generally known as settlement of foundation. Tilting, cracking and failure of superstructures may result from soil liquefaction and foundation settlement. Differential settlement may occur in conventional foundation system (individual column footing) due to excessive unbalance force induced by ground shaking. One footing failure may collapse number of footing and make the super structure to collapse. Differential settlement is smaller in continuous wall footing provided in confined method and within allowable limit. But in conventional isolated footing foundation, differential settlement is greater and exceed allowable limit sometimes. By providing continuous wall footing, differential settlement has kept within allowable limit as the reinforced concrete slab resists differential movements between loading positions and thus safety of the building to collapse due to earthquake force and vertical load has ensured.

For seismic resistance the building must be well connected to a good foundation and the earth. Also wet, organic soils should be avoided and the foundation must be well tied together as well as tied to the wall. It is good practice even in quite low structure, especially those founded on soft soils, to provide ties between column pads (Baidya, 2007). This tie action effectively resists the base shear. In the proposed foundation system, RC wall footing base will be constructed first; infill masonry wall will be constructed on footing base keeping space for columns. When masonry wall will reach up to the bottom of the grade beam, the columns will be constructed. Then grade beams will be constructed directly on masonry infill wall and columns. This construction sequence may be called “infill wall first; then RC frames” and generally known as confined masonry construction. So tie action obtained in continuous wall

footing by RCC base, masonry wall, short column. As thus the wall footing, columns and grade beams will act as an integrated or composite structure to resist the lateral loads induced by earthquake increasing stiffness of the foundation system. Thus the foundation system would be earthquake resistant.

With shallow foundations it is normal to assume that most of the resistance to lateral loads is provided by friction between the soil and the base of the members resisting horizontal load. As the footing area of the proposed foundation is greater than isolated column footing, larger frictional surfaces obtained in confined method which resisted greater lateral loads at foundation level.

As per BNBC (1993), equivalent static force method is applicable for low rise building for seismic analysis with regular structure. Seismic base shear is mainly dependent on fundamental period of structure, structural system modification factor and seismic dead load. Fundamental period and structural system modification factor are smaller in confined method rather than conventional bare frame method. Dead load variation is smaller. So obviously base shear calculated is greater in confined method. As a result, structure in confined method is analyzed and designed with maximum lateral loads and base shear rather than conventional method. So foundation of confined method (continuous wall footing) was designed by taking maximum base shear, maximum overturning moments to make the foundation better seismic resistant than conventionally designed isolated footing. Also shear resistance of confined infill wall in continuous wall foundation resist a considerable amount of base shear to make the foundation safer in earthquake.

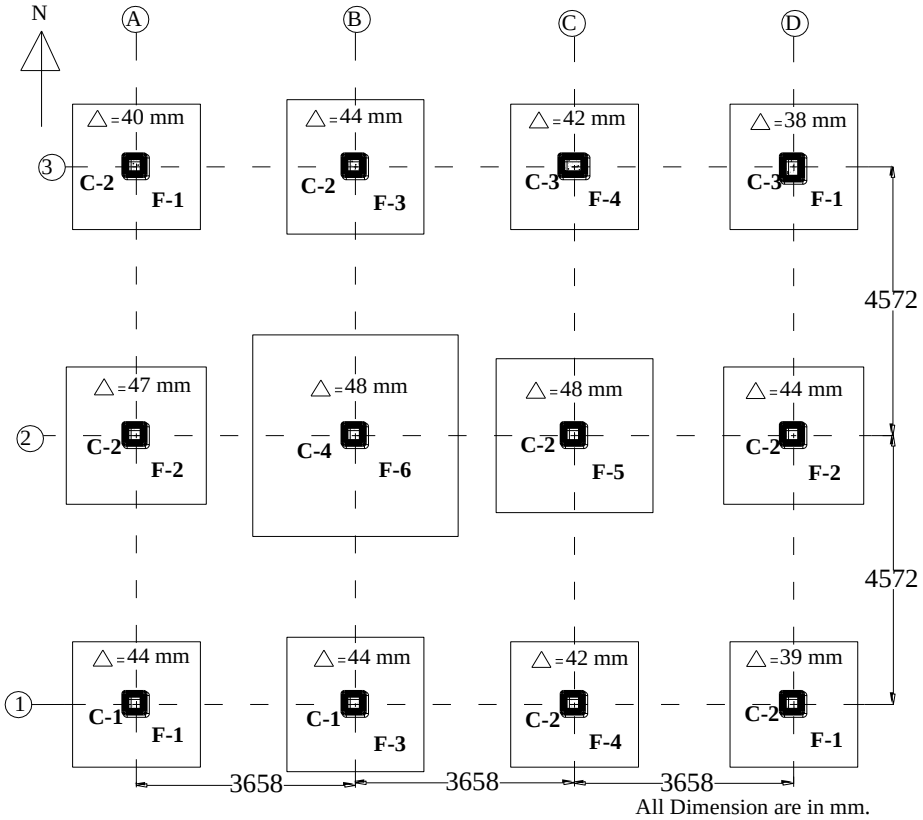
Sinking or tilting of individual foundations of columns due to seismic shaking occurs in loose soft soils and can lead to severe cracking of the superstructure and even collapse. But this risk can be eliminated by providing continuous footing. Thus the risk of Sinking or tilting affect of foundation is quite less in confined method.

Table 4.4: Area Spring Constant (K_s) of Conventional foundation in Local 3 direction

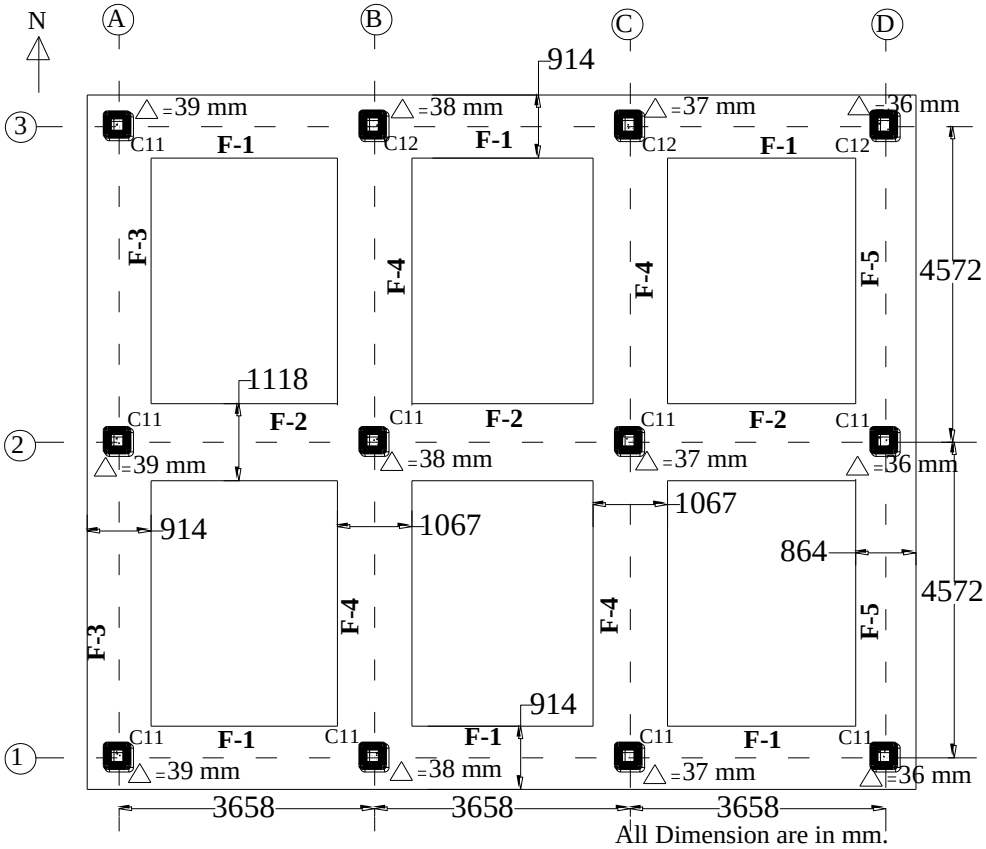
Footing Index	Allowable Bearing Capacity, q_a (kPa)			Selected settlement, Δ (mm) and Footing size, A (m x m)	Spring Constant, K_s (kN/m^3)
	Bearing capacity analysis	Settlement analysis	Selected Bearing capacity		
F1	95	84	84	Δ= 37 mm; A=2.1x2.1	2270
F2	95	78	78	Δ= 39 mm; A=2.3x2.3	2000
F3	95	80	80	Δ= 37 mm; A=2.2x2.2	2162
F4	95	82	82	Δ= 37 mm; A=2.1x2.1	2216
F5	95	71	71	Δ= 40 mm; A=2.6x2.6	1775
F6	95	49	49	Δ= 39 mm; A=3.4x3.4	1256

Table 4.5: Line Spring Constant (K_s) of Confined foundation in Local 2 direction

Footing Index	Allowable Bearing Capacity, q_a (kPa)			Selected settlement, Δ (mm) and Footing size, A(m x m)	Spring Constant, K_s (kN/m^2)
	Bearing capacity analysis	Settlement analysis	Selected Bearing capacity		
F1	95	92	92	Δ= 43 mm; A=0.9x11	1926
F2	95	96	95	Δ= 49 mm; A=1.1x11	2075
F3	95	90	90	Δ= 41 mm; A=0.9x9.15	1976
F4	95	89	89	Δ= 45 mm; A=1.1x9.15	2116
F5	95	86	86	Δ= 38 mm; A=0.86x9.15	1953



FOUNDATION SETTLEMENT (CONVENTIONAL METHOD)



FOUNDATION SETTLEMENT (CONFINED METHOD)

Figure 4.5: Foundation settlement of conventional and confined method

Figure 4.4 and 4.5 stated a clear picture that, settlement of foundation in confined method (continuous wall footing) is lower than that of conventional individual foundation. On an average foundation settlement is 14% lower in confined method where wall footing was provided rather than conventionally designed individual footing.

Differential settlement among points A3 to B3 in Confined method is=39-38=1 mm.
Differential settlement among points A3 to B3 in Conventional method is=44-40=4 mm which is 4 times higher than confined method.
Differential settlement among points C3 to D3 in Confined method is=37-36=1 mm.
Differential settlement among points C3 to D3 in Conventional method is=42-38=4 mm which is 4 times higher than confined method.
Differential settlement among points C2 to D2 in Confined method is=37-36=1 mm.
Differential settlement among points C2 to D2 in Conventional method is=48-44=4 mm which is 4 times higher than confined method.

So it has been proved that foundation settlement and differential settlement are lower in confined method (continuous wall footing) rather than that of conventional individual foundation. If settlement has occurred in foundation due to earthquake or gravity loads, total settlement and differential settlement will be lower if the building was designed and constructed in confined method by providing continuous wall footing than conventionally designed individual footing. Risk of foundation failure will be minimized as settlement is lower in proposed confined method. As foundation failure leads in super structure failure, minimizing risk of foundation failure also leads in minimizing risk of super structure failure. As thus proposed confined method easily resist earthquake effect. So continuous confined wall footing will be the most effective earthquake resistant foundation system for low rise building.

4.3.2 Earthquake resistance of foundation in soft pocket condition

If a soft pocket of soil exist in any panel beneath the foundation, structural effect is significant in conventional method than confined method. Also settlement and differential settlement is larger in conventional individual foundation than confined wall footing. To proof this, a separate model was prepared assuming the panel A3-B3-A2-B2 as a soft clay pocket in the foundation which compression index, C_c=0.2 and all other panel having compression index, C_c=0.10. Bearing capacity and settlement analysis was carried on to determine the allowable bearing capacity, footing size and settlement. Trial and error method was utilized to determine spring constant or modulus of subgrade reaction (K_s), allowable bearing capacity, settlement and footing size. By putting the selected footing size and spring constant in the model, static analysis was carried on in ETABS.

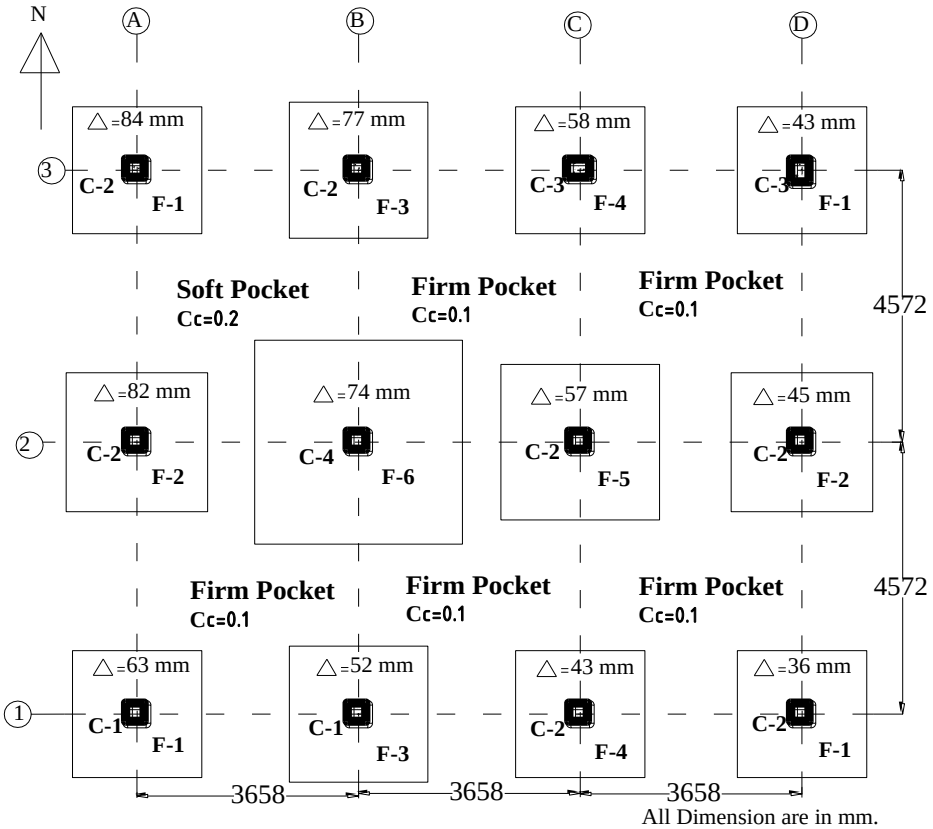
Table 4.6: Area Spring Constant (K_s) of Conventional foundation in Local 3 direction for soft pocket condition

Footing Index	Allowable Bearing Capacity, q_a (kPa)			Selected settlement, Δ (mm) and Footing size, A (m x m)	Spring Constant K_s (kN/m^3)
	Bearing capacity analysis	Settlement analysis	Selected Bearing capacity		
F1(A-3)	95	80	80	Δ= 71 mm; A=2.1x2.1	1127
F3(B-3)	95	78	78	Δ= 73 mm; A=2.2x2.2	1068
F2(A-2)	95	64	64	Δ= 68 mm; A=2.3x2.3	941
F6(B-2)	95	33	33	Δ= 55 mm; A=3.4x3.4	600
F1	95	113	95	Δ= 47 mm; A=2.1x2.1	2021
F2	95	72	72	Δ= 37 mm; A=2.3x2.3	1946
F3	95	90	90	Δ= 41 mm; A=2.2x2.2	2195
F4	95	108	95	Δ= 45 mm; A=2.1x2.1	2111
F5	95	80	80	Δ= 47 mm; A=2.6x2.6	1702

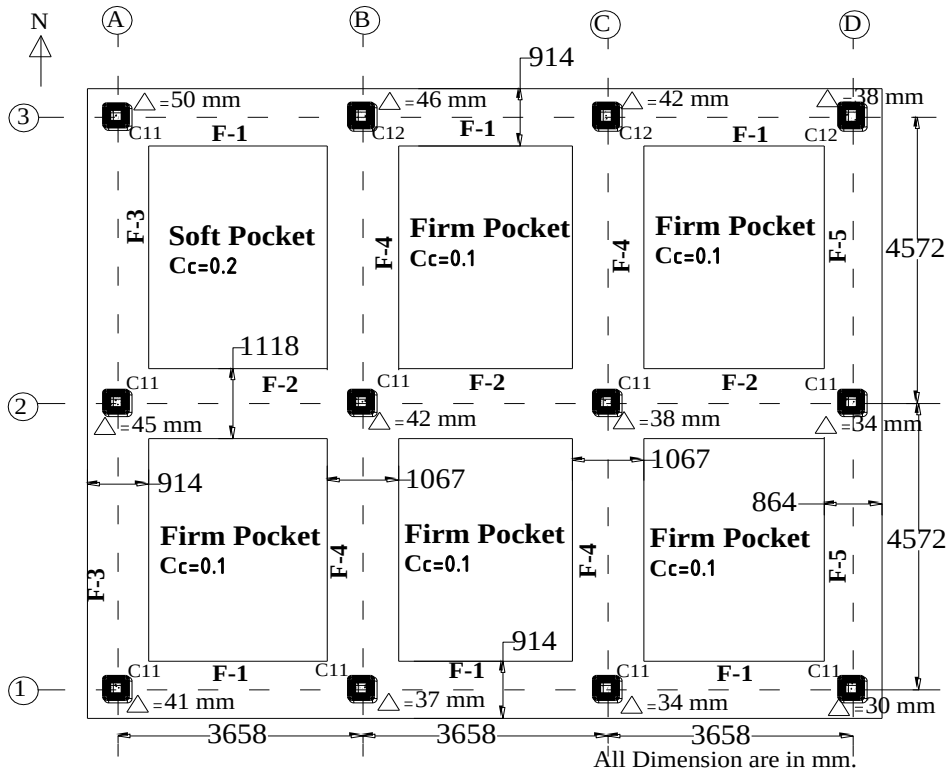
Table 4.7: Line Spring Constant (K_s) of Confined foundation in Local 2 direction for soft pocket condition

Footing Index	Allowable Bearing Capacity, q_a (kPa)			Selected settlement, Δ (mm) and Footing size, A (m x m)	Spring Constant K_s (kN/m^2)
	Bearing capacity analysis	Settlement analysis	Selected Bearing capacity		
F1 (A-B)/3	95	98	95	Δ= 65 mm; A=0.91x3.66	1330
F2 (A-B)/2	95	106	95	Δ= 77 mm; A=1.1x3.66	1332
F3 (2-3)/A	95	86	86	Δ= 64 mm; A=0.91x4.57	1223
F4 (2-3)/B	95	82	82	Δ= 72 mm; A=1.16x4.57	1321
F1 (B-D)/3	95	96	95	Δ= 41 mm; A=0.916x7.3	2109
F2 (B-D)/2	95	87	87	Δ= 43 mm; A=1.16x7.32	2185
F1	95	84	84	Δ= 41 mm; A=0.91x11	1864
F3 (1-2)	95	98	95	Δ= 35 mm; A=0.91x4.57	2470
F4 (1-2)	95	81	81	Δ= 36 mm; A=1.16x4.57	2610
F4 (1-3)	95	85	85	Δ= 46 mm; A=1.16x9.15	2143
F5	95	83	83	Δ= 37 mm; A=0.86x9.15	1929

Figure 4.6 stated a clear picture that, settlement of foundation in confined method (continuous wall footing) is lower than that of conventional individual foundation if soft pocket exist beneath the foundation. On an average foundation settlement is 33% lower in confined method where wall footing was provided rather than conventionally designed individual footing.



FOUNDATION SETTLEMENT (CONVENTIONAL METHOD)



FOUNDATION SETTLEMENT (CONFINED METHOD)

Figure 4.6: Foundation settlement of conventional and confined method for soft soil

Differential settlement among points A3 to D3 in Confined method is=50-38=12 mm. Differential settlement among points A3 to D3 in Conventional method is=84-43=41 mm which is 3.4 times higher than confined method.

Differential settlement among points A2 to D2 in Confined method is=45-34=11 mm. Differential settlement among points A2 to D2 in Conventional method is=82-45=37 mm which is 3.36 times higher than confined method.

Differential settlement among points A1 to D1 in Confined method is=41-30=11 mm. Differential settlement among points A1 to D1 in Conventional method is=63-36=27 mm which is 2.5 times higher than confined method.

So it has been proved that, if a soft pocket of soil exist in any panel beneath the foundation, for the present case settlement 33% and differential settlement 300% larger in conventional individual foundation than confined wall footing.

If a soft pocket of soil exist in any panel beneath the foundation, structural effect is significant in conventional method than confined method. Figure 4.7 shows that, reinforcement percentage and member size is higher in conventional method than confined when a soft pocket exists beneath the foundation.

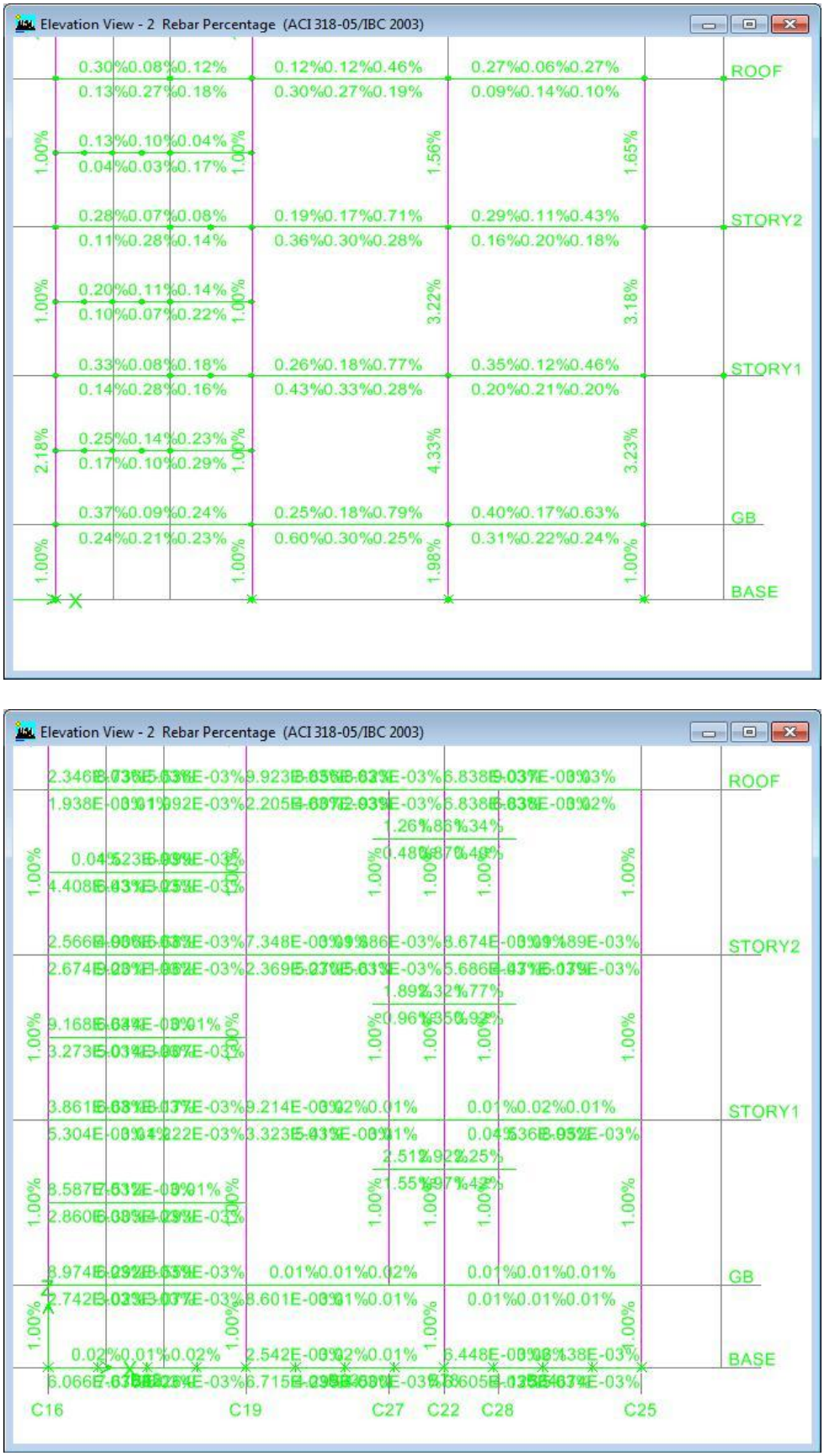


Figure 4.7: Rebar % of conventional and confined method for soft pocket condition

4.3.3 Lateral Resistance, Storey Shear Capacity and Deformability

Building configuration, opening for door & windows, rigidity distribution, ductility, construction quality also affect the seismic resistance capacity of the building.

Building configuration means building shape. The concerned building is rectangular in shape and length to width ratio is less than 3.0 as per several code provisions for seismic resistance. As per eurocode8, opening size kept smaller than 1.5 m2 to resist efficient seismic load. The rigidity of the building in the vertical direction has kept uniform without changing any structural system. All columns have been run from foundation to top floor of the same size in the confined method. Construction quality has ensured by adopting the system confined method. By adopting confined method of construction inadequate bonding of masonry wall with RC frame structure has denied, so proper bonding increases the seismic resistant capacity of the building and stiffness.

For better seismic performance all elements, such as walls and the roof, should be tied together so as to act as an integrated unit during earthquake shaking, transferring forces across connections and preventing separation. So in confined method wall and roofs has tied together by frame element like column, beam and lintel. Also infill wall has modeled and designed as per real existence in the building, so confined method measures actual behavior of the RC frame structured building having infill wall during earthquake whereas in conventional method infill brick wall load is distributed on beams and slabs only, no modeling of infill wall has done and thus this method does not measures real behavior of the building having infill wall. So building will be better seismic resistant and stiffer if real modeling and structural design has done.

To safe the building from deformability, components must have proper aspect ratios (that is not be too high), must be adequately connected to resisting elements (for example sufficient wall ties for a masonry wall, tying it to floors, roof and shear walls) and must be well tied together so as to permit large deformations and dynamic

motions to occur without sudden collapse and to resist shear failure. Deformability requires that structures be well-proportioned, regular and well tied together so that excessive stress concentrations are avoided and forces are capable of being transmitted from one component to another even through large deformations whish has achieved in confined method but not conventional bare frame method. To satisfy damageability, quality of construction of masonry wall has maintained by using 1st class burnt clay bricks and mortar ratio of 1:3 (Cement:Sand) and good quality concrete of ratio 1:2:3 (cement:sand:chips).

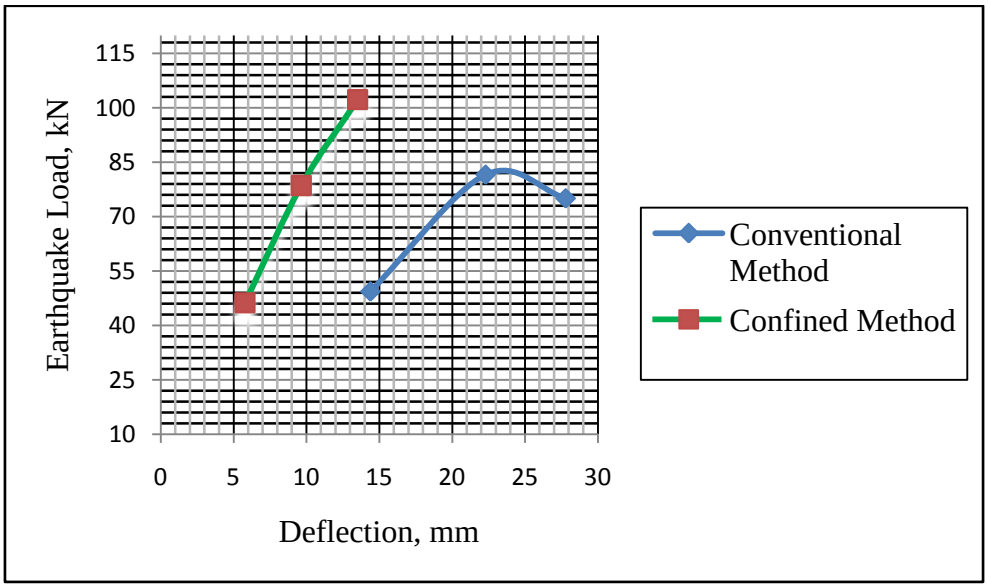


Figure 4.8: Load deflection characteristic for the Conventional and Confined Method (YY Axis)

Figure 4.8 shows that, approximately in the same lateral load for the storey 1 and 2, deflection is 40% and 43% lower in confined method rather than conventional method respectively. These variations were found more in storey 3 (roof level).

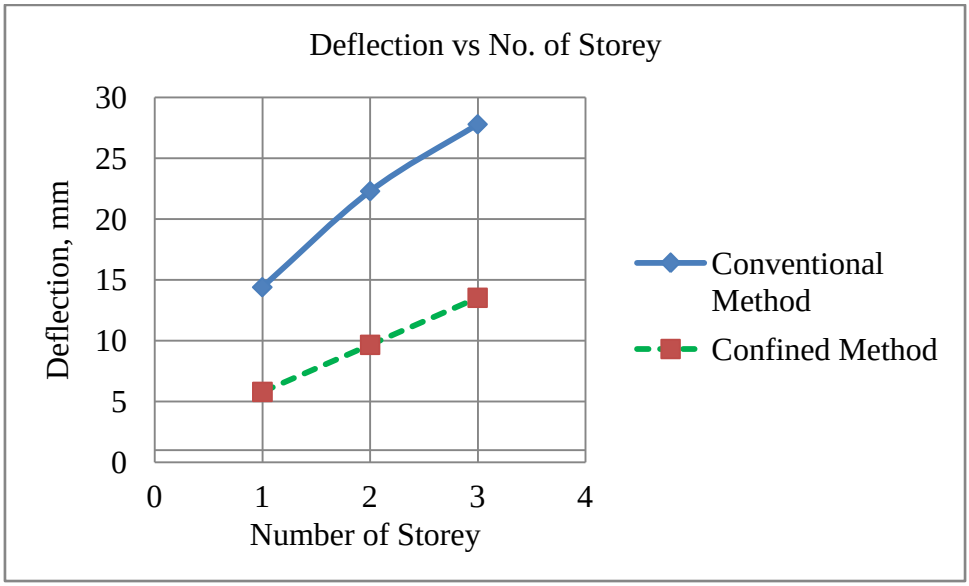


Figure 4.9: Deflection characteristic with Number of Storey in both Methods (YY Axis)

Figure 4.9 shows that, deflection is increased with the number of storey, but in storey 3 deflection is 44.13% smaller in confined method rather than conventional bare frame method.

Table 4.8: Storey shear Force, Storey Displacement and Stiffness of the Buildings in XX direction

XX Direction									
Storey	H (m)	Conventional Method T=0.45 S			Confined Method T=0.169 S			Difference (%) ΔT=62.44%	
		Q kN	δ mm	K kN/mm	Q kN	δ mm	K kN/mm	δ %	K %
3	3.05	74.84	22.9	3.3	102.9	10.31	9.98	45.0	33.06
2	3.05	156.4	17.8	8.78	180.9	7.38	24.51	41.5	35.82
1	3.05	205.4	11.6	17.71	226.8	4.43	51.2	38.19	34.59

Table 4.9: Storey shear Force, Storey Displacement and Stiffness of the Buildings in YY direction

YY Direction									
Storey	H (m)	Conventional Method T=0.45 S			Confined Method T=0.145 S			Difference (%) ΔT=31.6%	
		Q kN	δ mm	K kN/mm	Q kN	δ mm	K kN/mm	δ %	K %
3	3.05	74.84	27.8	2.69	102.9	13.54	7.60	48.7	35.4
2	3.05	156.4	22.3	7.01	180.9	9.67	18.71	43.4	37.5
1	3.05	205.4	14.4	14.26	226.8	5.8	39.10	40.3	36.5

From Table 4.8 and 4.9, it has been found that storey displacement is less than an average of 41.5% in XX direction and an average of 44.13% in YY direction in confined method rather than conventional bare frame method. Also deflection in both the method is within limit value of 0.005h as per BNBC-1993 where h is the height of the building. In Confined method Drift ratio in YY and XX direction are 0.15% and 0.11%. Experimental study on 03 storied confined masonry building shows that first diagonal cracking occurred at a drift ratio to 0.36%, crack propagation into the tie column ends, thus shearing off these elements, was recorded at a drift ratio to 0.67% (Alcocer, Arias and Vazquez, 2004) which are well below found in the proposed confined method. Sattar and Liel (2010) found that ultimate load occurred in infilled frame at a drift ratio of 0.22%.

Table 4.8 and 4.9, show that storey shear capacity is 21% greater in confined method than conventional method. Experimental study shows that storey shear strength is 13% greater in confined masonry construction rather than conventional bare frame method and typical drift ratio at failure were also improved significantly (Agha, Yingmin, salih, Zoubi and Saeed, 2010).

4.3.4. Lateral Stiffness, Ductility, and Energy dissipation capacity

Confined masonry infill wall panels increase strength, stiffness, overall ductility and energy dissipation capacity of the building. Experimental study shows that "initial stiffness of the infilled frame is 10 times greater than that of convention bare frame; yield strength of infilled frame is 2 times greater than that of conventional bare frame" (Murthy and Nagar, 1996). Reduction in stiffness is due to the improper bonding of infill wall and RC frame. In conventional method, RC frame is constructed first and then infill brickwall is bonded with frame, but gap found in between brick wall and beam at top of the wall in most cases due to construction sequence. These gaps allow the beams to deflect to some extent without transferring the gravity loads to the wall below. Thus this wall does not survive during ground shaking by earthquake due to the gap of infill and frame, so out of plane and sometimes in plane failure occurred. But in confined method, directly casting an upper beam on the infill wall creates better integrity between the frame and infill and no gap found between wall and frame. Thus stiffness of the building in confined method is larger than conventional RC frame method.

Figure 4.10: Stiffness characteristic for the Conventional and Confined Method (YY Axis)

Figure 4.10 shows that in each storey, stiffness of the storey is larger in Confined method rather than Conventional method. Table 4.8 and 4.9, show that stiffness is greater than an average of 34.5% in XX direction and an average of 36.5% in YY direction in confined method rather than conventional bare frame method. Experimental study shows that confined method increased the lateral stiffness at the maximum load level by 11% as compared with conventional bare frame method (Agha, Yingmin, salih, Zoubi and Saeed, 2010). Another scaled model test show that confined infill wall increasing the strength by 100% and stiffness by 500% of the bare RC frame structure building (Pujol, et al, 2008).

Ductility is the most desirable quality for good earthquake performance. To satisfy ductility demand, all seismic resistant detailing of structure for intermediate moment resisting frame (Seismic design category C) as per BNBC-1993 has been followed in both methods. The beams and column undergo less deformation and displacement owing to the presence of infill panel and are able to sustain the earthquake forces with less damage. The yield displacement of infilled frames is much smaller than that of the bare frame, and hence, the infilled frames have a considerable larger ductility.. Infill wall in confined method reduces member ductility demand which is largely required in conventional method to make the building earthquake resistant. When earthquake loads applied in buildings with confined method, masonry infill also absorb unbalance energy with the frame to make weak the earthquake (Mangkoesoebroto, GOTO and Tambunan, 2001), so frame members required less ductility to withstand earthquake. But in conventional method infill wall does not absorb (sometimes very little absorb) energy to weak the earthquake, only frame members absorb energy, so members have to be more ductile to withstand earthquake. As thus the presence of infill wall significantly reduces the maximum displacements and member ductility demand to withstand lateral loads.

Experimental study shows that, the average ductility of unreinforced masonry (URM) infilled frames is about 4.0 times that of the bare frames and URM infilled frames have about 70% higher strength than the bare frames (Murthy and Sudhir, 2000). Experimental study also shows that ductility in the confined masonry wall was

increased from 79% to 186% rather than bare frame (Agha, Yingmin, salih, Zoubi and Saeed, 2010).

The energy dissipation is affected by the confining elements like column and beams. At the deformation level of displacement, all confined wall dissipated far more energy than the unconfined wall. Experimental study shows that in confined method energy dissipation capacity is 89% higher rather than conventional frame method (Agha, Yingmin, salih, Zoubi and Saeed, 2010).

Results of pushover analysis show an increase in initial stiffness, strength, and energy dissipation of the infilled frame, compared to the bare frame, despite the wall's brittle failure modes. Likewise, dynamic analysis results indicate that fully-infilled frame has the lowest collapse risk and the bare frames are found to be the most vulnerable to earthquake-induced collapse (Sattar and Liel, 2010). Seismic resistance of confined masonry buildings has been experimentally verified by Tomazevic and Klemenc, (1997).

CHAPTER 5
CONCLUSIONS

5.1 General

Shallow foundations are often of a form that is highly vulnerable to damage from differential horizontal and vertical ground movements during earthquakes. This risk is greater in conventional isolated column footing where footings are separated to each other. It is therefore good practice even in quite low structure, especially those founded on soft soils, to provide ties between column pads. Continuous masonry wall footing is the best, easy to construction and economic solution to satisfy tie criteria. The tie action obtained by constructing confined masonry infill, base, columns and grade beams. The presence of masonry infill wall change the response of the building under seismic action and it is essential to consider infill walls for seismic evaluation of the structure. Finite element linear static analysis was performed utilizing the resistance of masonry infills in RC frame structure to make the foundation and structure earthquake resistant. By adopting Confined method in modeling, structural design and construction of low rise building, stiffness, strength, overall ductility, energy dissipation etc. can be increased to great extent; also it can decrease top deflection, settlement of foundation and vulnerability of foundation and structure to earthquake failure rather than Conventional bare frame method.

5.2 Conclusions

The following conclusions may be drawn with respect to this numerical and analytical study:

i. Affect of confined masonry infill wall in foundation design and construction increases the seismic resistance and strength of the foundation for low rise building.

ii. Low rise building designed and constructed in Confined method and with continuous wall footing predominately decreases the possibility the differential settlement of foundation in firm and soft soil due to gravity and earthquake load.

Foundation settlement is 14% lower and differential settlement is 4 times smaller in proposed confined method than conventional method.

iii. Low rise building designed and constructed in Confined method and with continuous wall footing increases the stiffness of foundation by the integral group action or composite action of continuous wall footing, columns, and grade beams.

iv. Continuous wall footing in confined method eliminates the sinking and tilting of foundation during earthquake induced ground shaking rather than conventional method where isolated footing was provided.

v. If a soft pocket exists beneath the foundation, total settlement is 33% lower and differential settlement is 3 times smaller in proposed confined method than conventional method. Also structural effect is significant in conventional method. Reinforcement percentage and member sizes are larger in conventional method than confined method due to the effect of soft pocket existence.

vi. Stiffness of the low rise building designed and constructed in Confined method is greater an average of 34% in XX direction and 36% in YY direction in comparison with conventional bare frame method.

vii. Story displacement or top deflection of the low rise building designed and constructed in Confined method is smaller an average of 41% in XX direction and 44% in YY direction in comparison with conventional bare frame method.

viii. Foundation cost reduces approximately 21.5% by employing confined method with continuous wall footing rather than conventional method where individual column footing was employed for low rise building.

ix. Structural cost reduces approximately 23% by employing confined method rather than conventional method for 3-Storied low rise RC frame structured building.

x. Total project cost reduces approximately 15% by employing confined method rather than conventional method for 3-Storied low rise RC frame structured building.

xi. Opening size in infill has significant contribution towards fundamental period, inter storey drift ratio, infill stress and the structural member forces. So any specific stabilized code (like eurocode8) has to follow to limit the opening amount.

xii. Guideline for Confined masonry design and construction described in previous chapter can be included in Bangladesh National Building Code (BNBC) to its any later version than BNBC-1993 to form seismic design guide for low rise confined masonry building.

5.3 Recommendations for Future Study

From the lessons of the present study, the recommendations for future study may be summarized as follows:

i. Non linear dynamic analysis or non linear static analysis (pushover) can be performed for study the behavior of masonry infilled RC frame structured building in confined method and conventional bare frame method.

ii. Non linear dynamic analysis or non linear static analysis (pushover) can be performed for study of the effect of masonry infill wall on the progressive collapse potential of a RC building of confined method.

iii. Experimental testing of Finite element model of low rise building in confined method by shaking table and other instrument to verify the numerical study results. Conventional model can also be tested.

iv. Finite element modeling and design of conventional foundation (isolated column footing) and comparison with continuous wall footing by taking confined and conventional design methodology.

v. This study can be review by taking the soil properties from the sub-soil investigation for a particular area of Bangladesh.

vi. This study can be review by taking the masonry infill properties like compressive strength, Modulus of elasticity, Poisson’s ratio, mass from the brick prism test results and other laboratory test.

vii. Design methodology of RC frame structured building having infill wall in confined method with soft story effect in ground floor in aspect of Bangladesh under earthquake loading.

viii. Assessment and experimental study of strength associated with the out of plane collapse of masonry infill wall in RC frame structured low and medium rise building under seismic loading.

ix. Liquefaction and settlement analysis can be workout jointly to investigate foundation behavior on soft or weak soil.

x. Instead of masonry infill, other types of infill such as concrete block, hollow clay tile can also be considered for such type of investigation.

xi. Seismic repair and retrofit techniques of confined masonry construction in respect of Bangladesh.

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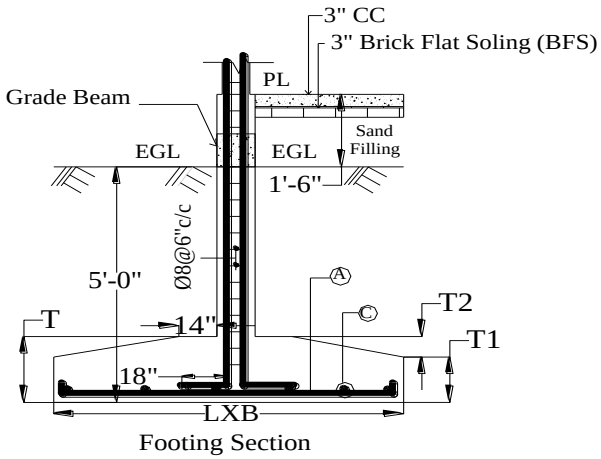
Appendix A

DESIGN DRAWINGS

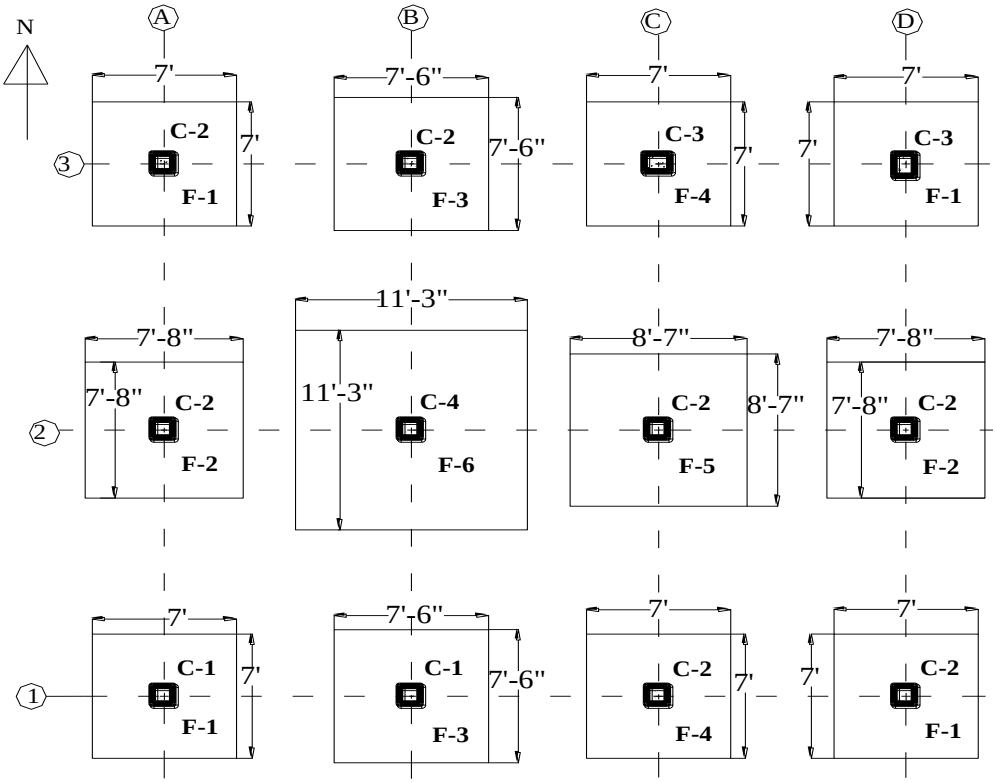
Design Sections of Conventional Method:

FOOTING SCHEDULE (CONVENTIONAL METHOD) :

SYMBOL	FOOTING SIZE L X B	THICKNESS			REINFORCEMENT IN SHORT DIRECTION (A)	REINFORCEMENT IN LONG DIRECTION (C)
		T1	T2	T		
F1	7'-0"x7'-0"	9"	4"	13"	9-16mmØ@9.75"C/C	9-16mmØ@9.75"C/C
F2	7'-8"x7'-8"	10"	4"	14"	10-16mmØ@9.5"C/C	10-16mmØ@9.5"C/C
F3	7'-6"x7'-6"	9"	4"	13"	9-16mmØ@10"C/C	9-16mmØ@10"C/C
F4	7'-0"x7'-0"	8"	4"	12"	8-16mmØ@11"C/C	8-16mmØ@11"C/C
F5	8'-7"x8'-7"	11"	4"	15"	13-16mmØ@8"C/C	13-16mmØ@8"C/C
F6	11'-3"x11'-13"	12"	4"	16"	18-16mmØ@7.5"C/C	18-16mmØ@7.5"C/C



Footing Section



FOOTING LAYOUT PLAN (CONVENTIONAL METHOD)

Figure A.1: Foundation Layout and Schedule

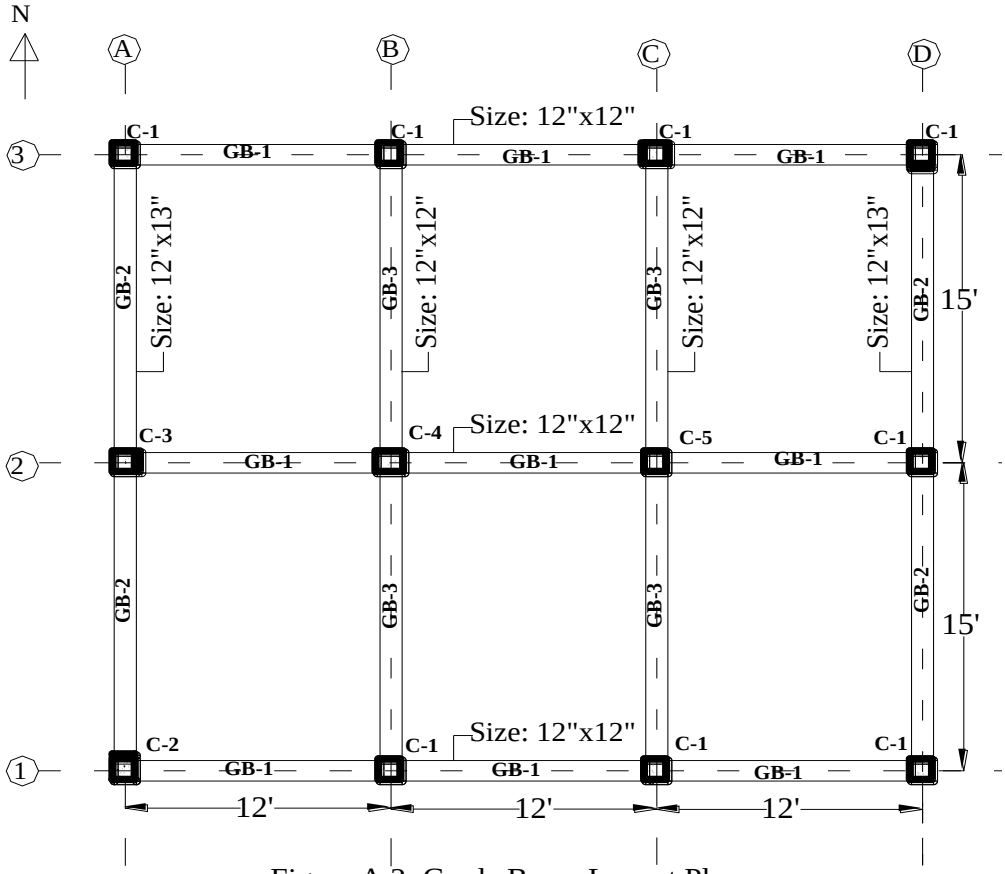


Figure A.2: Grade Beam Layout Plan

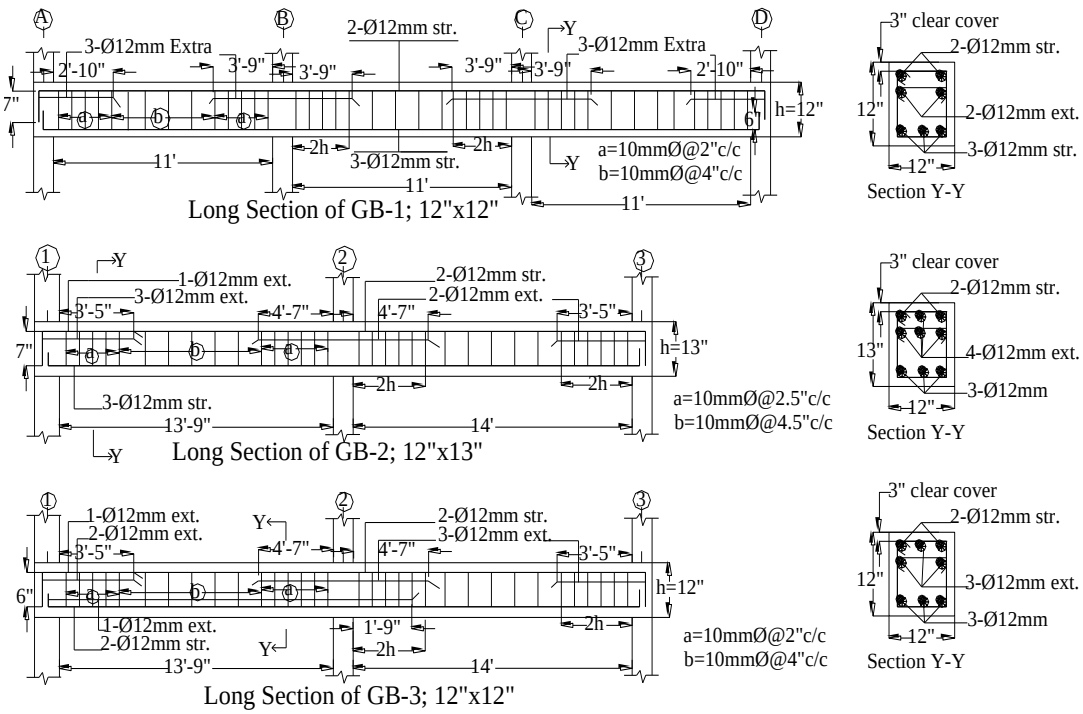
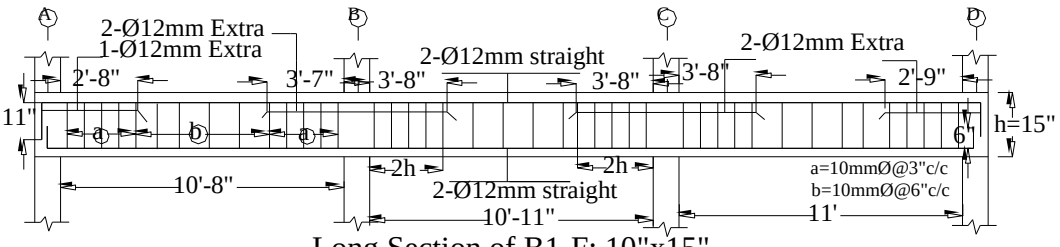
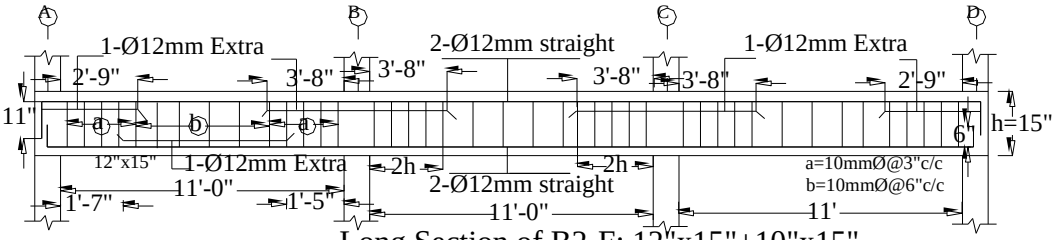


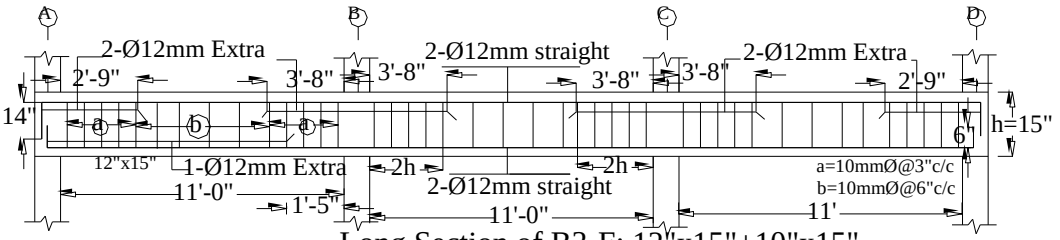
Figure A.3: Grade Beam Design sections



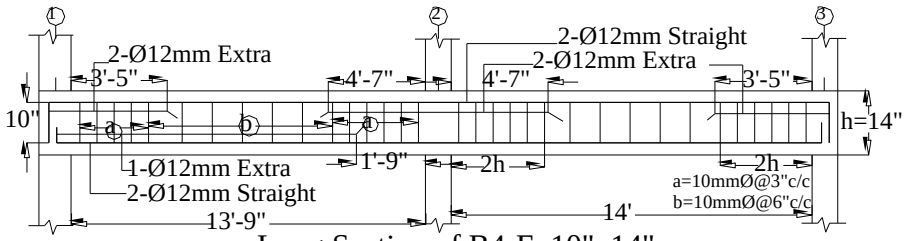
Long Section of B1-F; 10"x15"



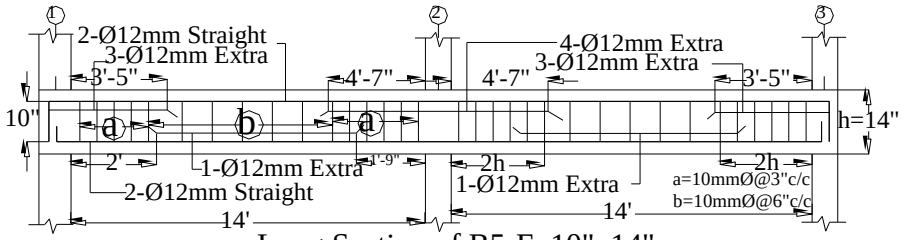
Long Section of B2-F; 12"x15"+10"x15"



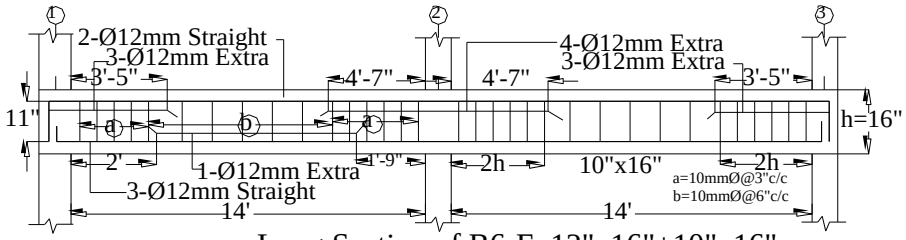
Long Section of B3-F; 12"x15"+10"x15"



Long Section of B4-F; 10"x14"



Long Section of B5-F; 10"x14"



Long Section of B6-F; 12"x16"+10"x16"

Figure A.4: Floor Beam Design sections

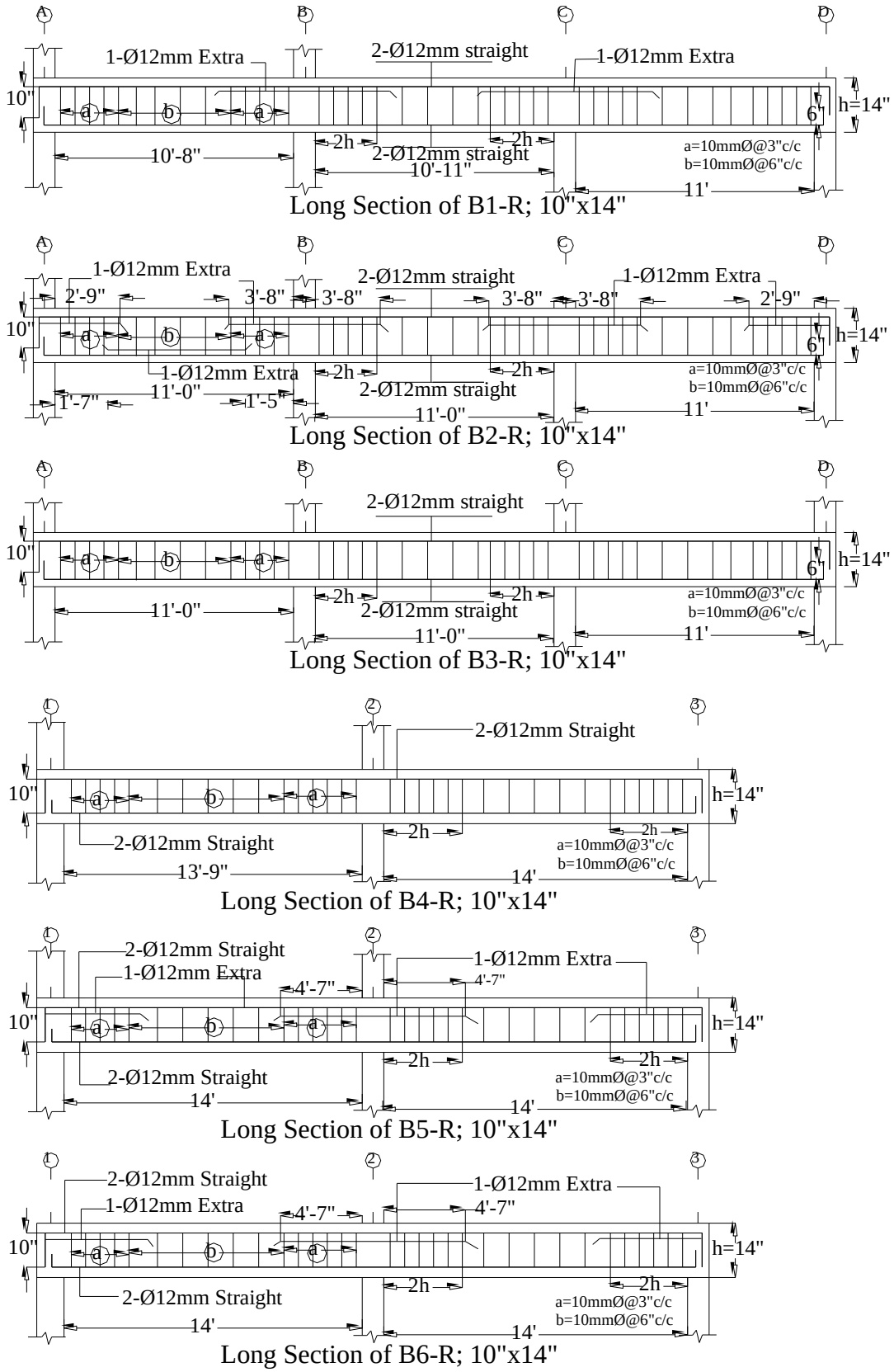


Figure A.5: Roof Beam Sections

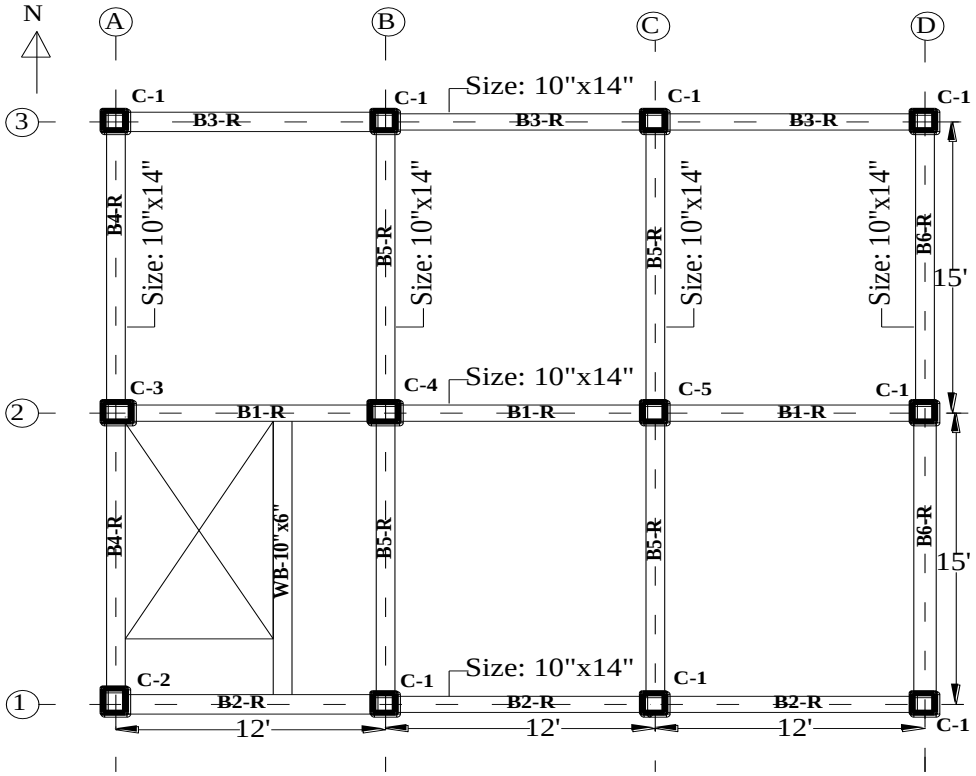


Figure A.6: Roof Beam Layout Plan

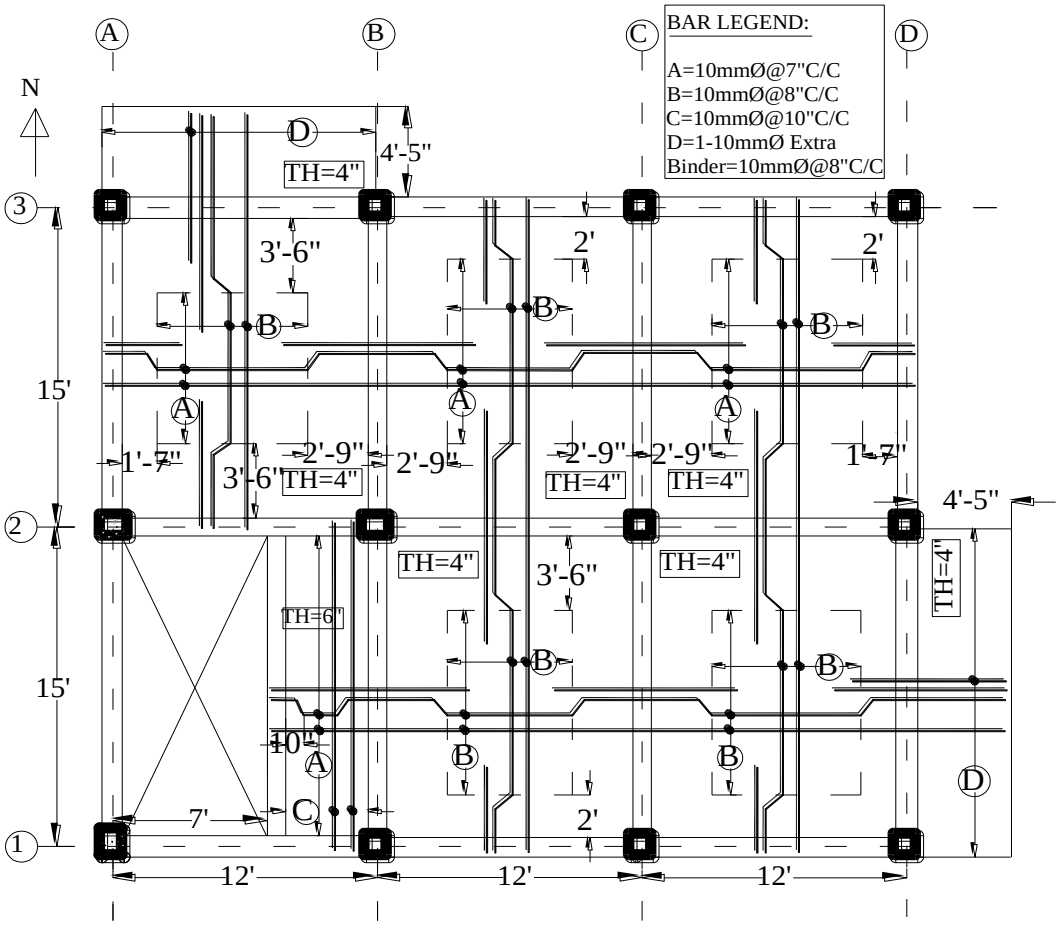


Figure A.7: Floor Slab Design

Design Sections of Confined Method:

FOOTING SCHEDULE (CONFINED METHOD) :

SYMBOL	FOOTING SIZE WIDTH, B	THICKNESS T	REINFORCEMENT IN SHORT DIRECTION (A)	REINFORCEMENT IN LONG DIRECTION (C)
F1	3'-0"	6"	10mmØ@7"C/C.	10mmØ@10"C/C.
F2	3'-8"	7"	10mmØ@6"C/C.	10mmØ@8.5"C/C.
F3	3'-0"	6"	10mmØ@7"C/C.	10mmØ@10"C/C.
F4	3'-6"	6"	10mmØ@5"C/C.	10mmØ@10"C/C.
F5	2'-10"	6"	10mmØ@8.5"C/C.	10mmØ@10"C/C.

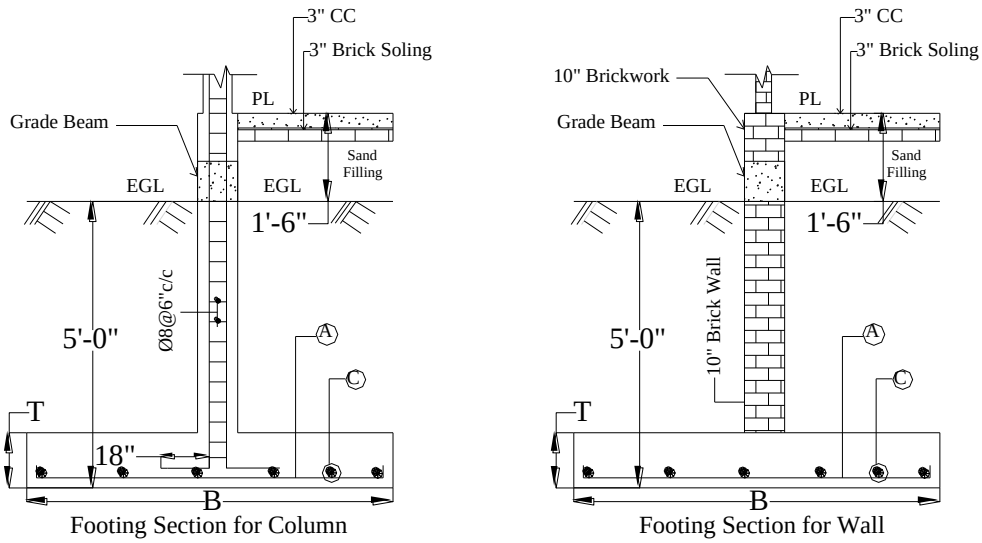


Figure A.8: Foundation Layout Plan

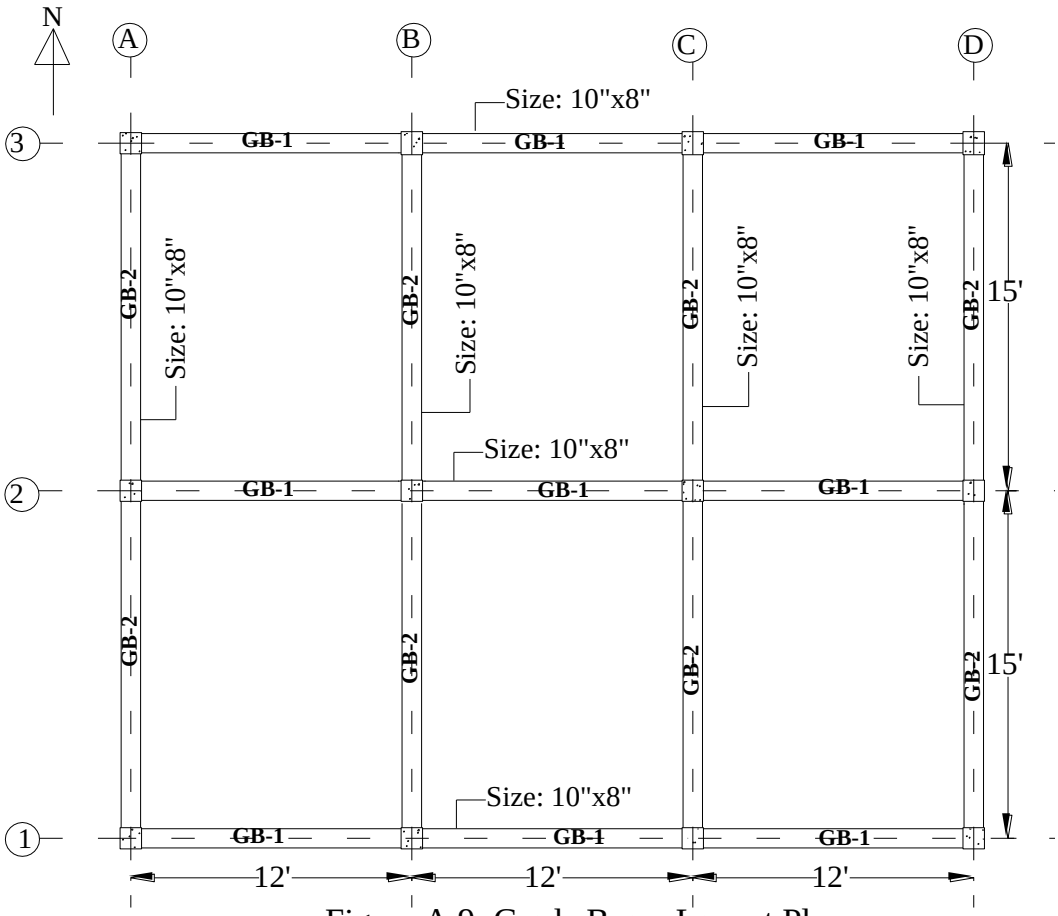


Figure A.9: Grade Beam Layout Plan

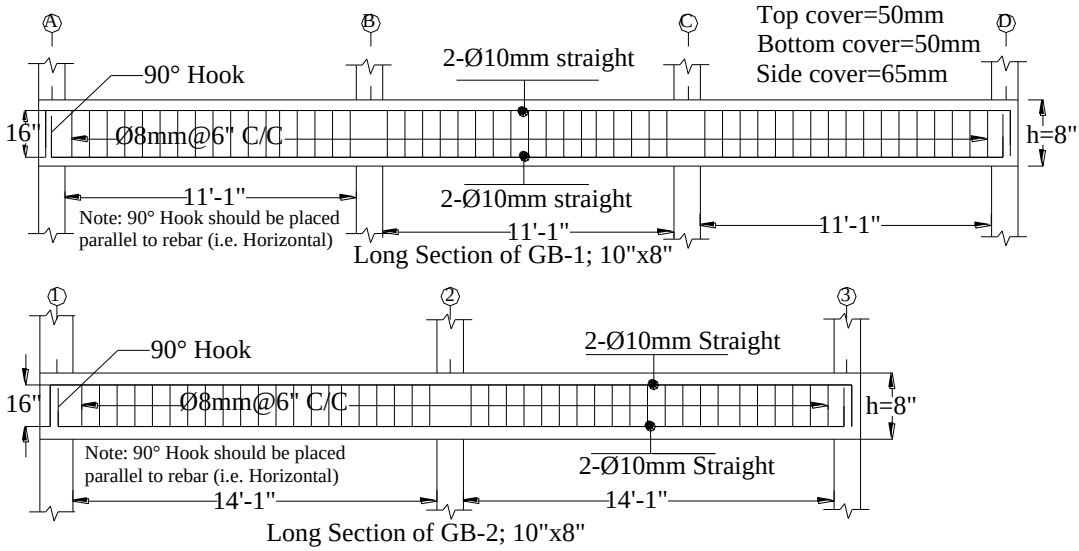


Figure A.10: Grade Beam Long Sections

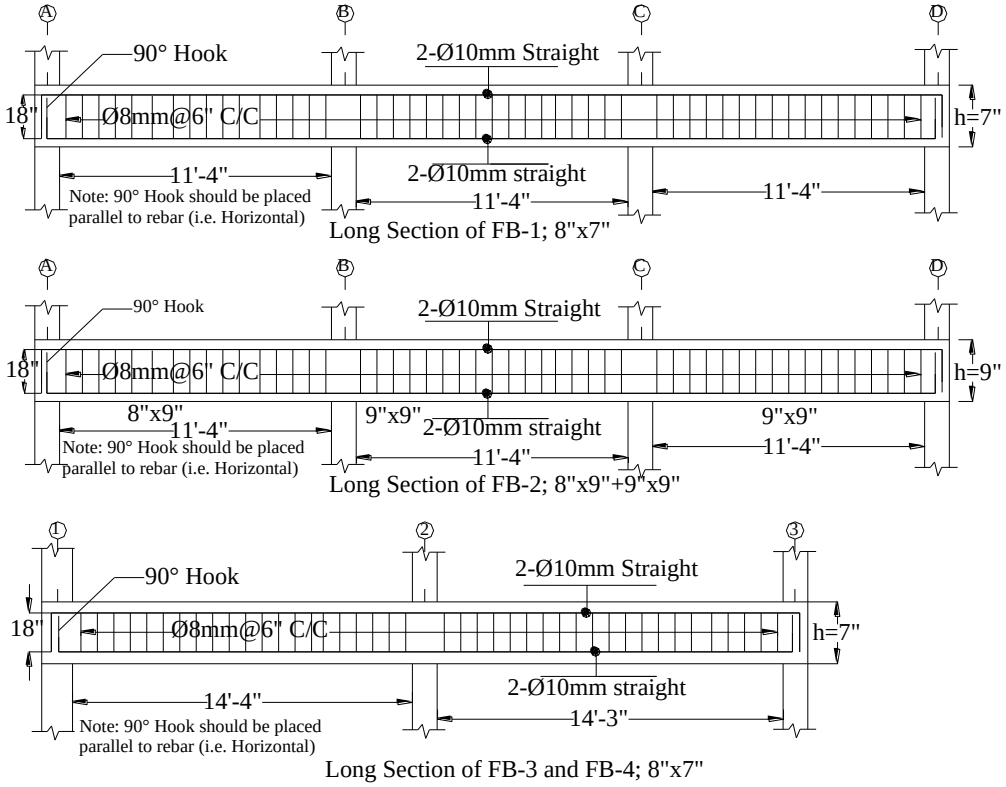


Figure A.13: Floor Beam Long Sections

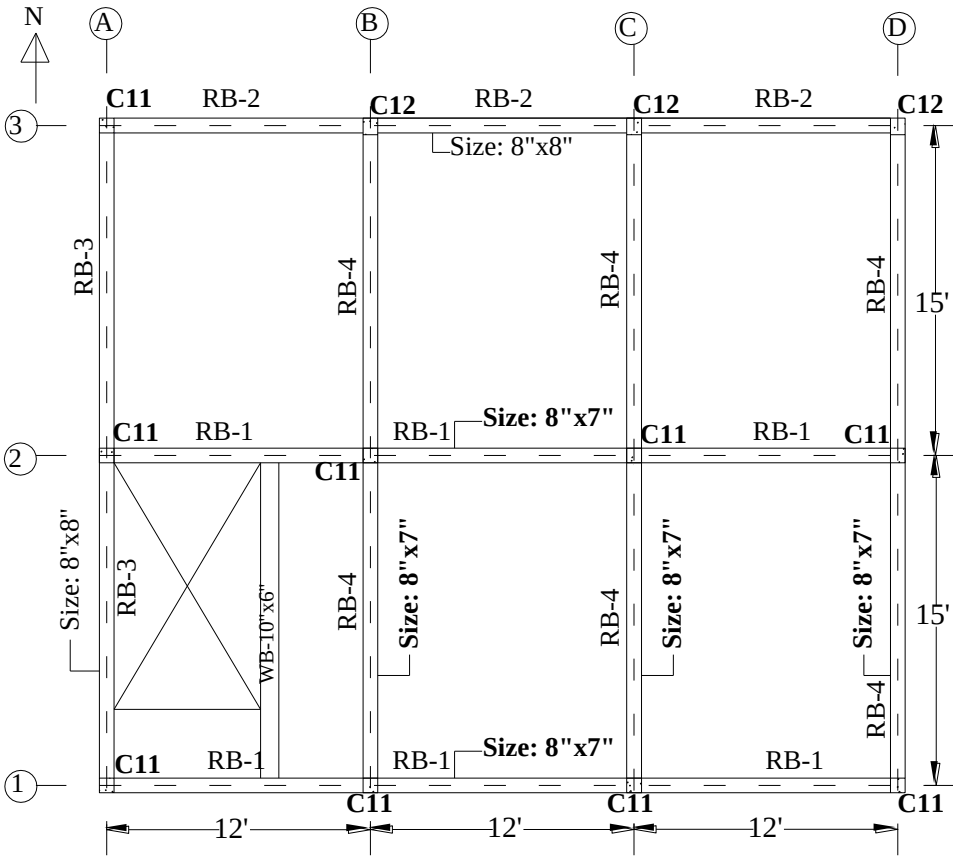


Figure A.12: Roof Beam layout Plan

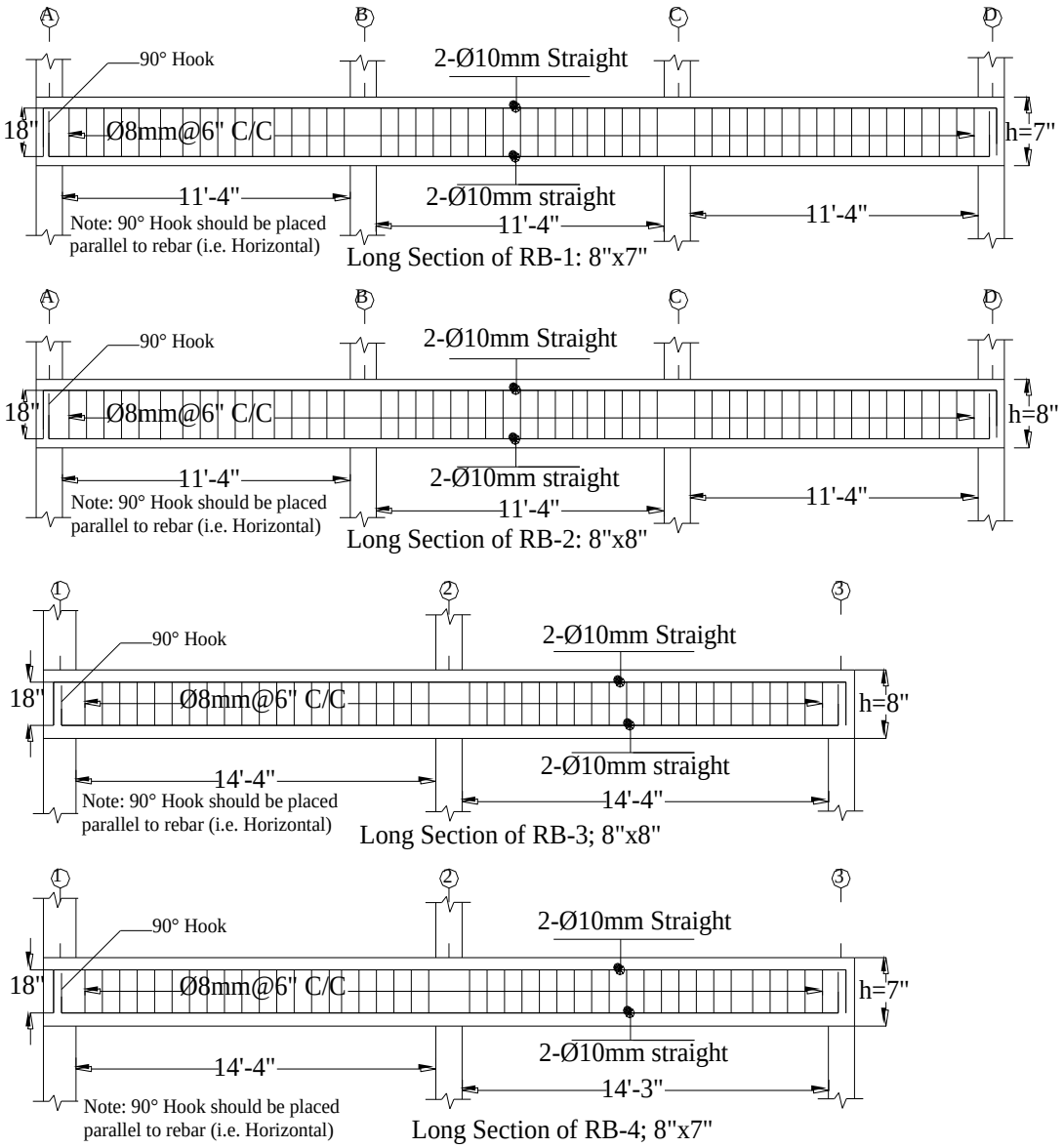


Figure A.13: Roof Beam Long Sections

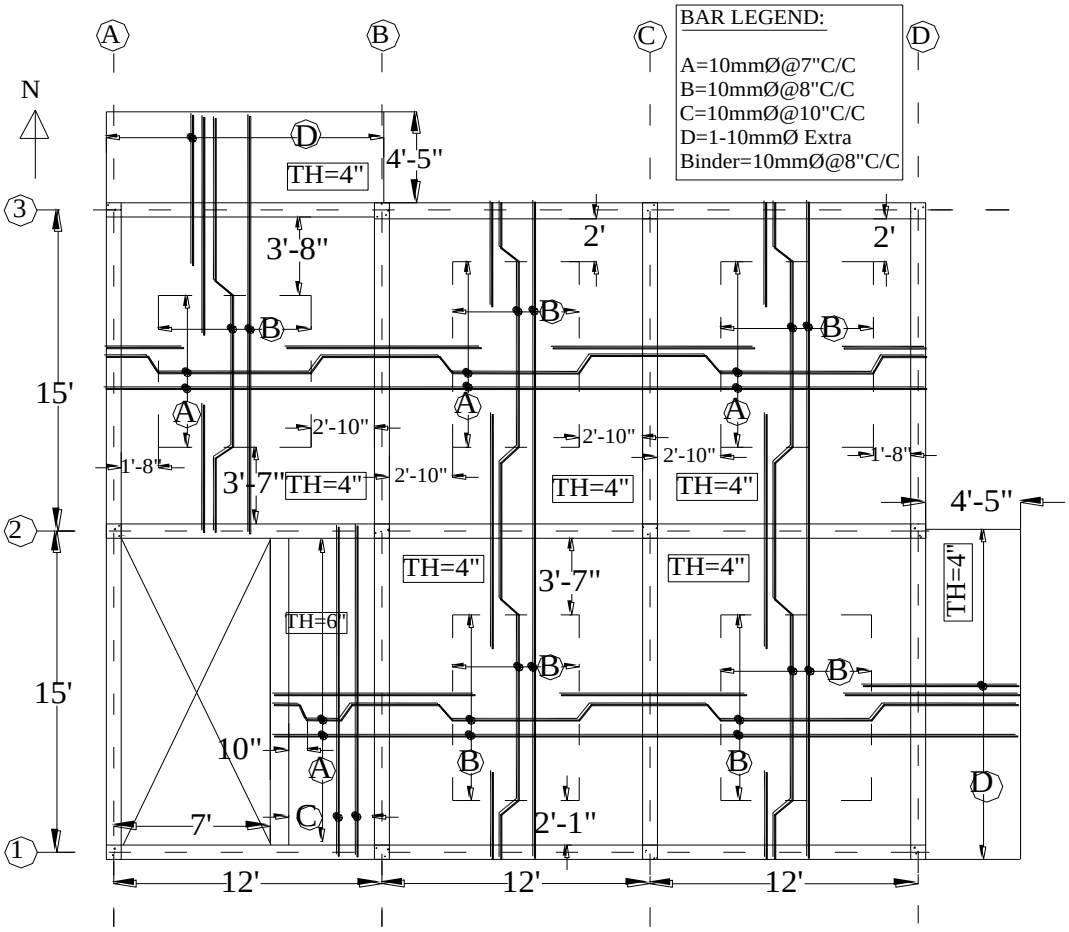


Figure A.14: Floor Slab Rebar Plan

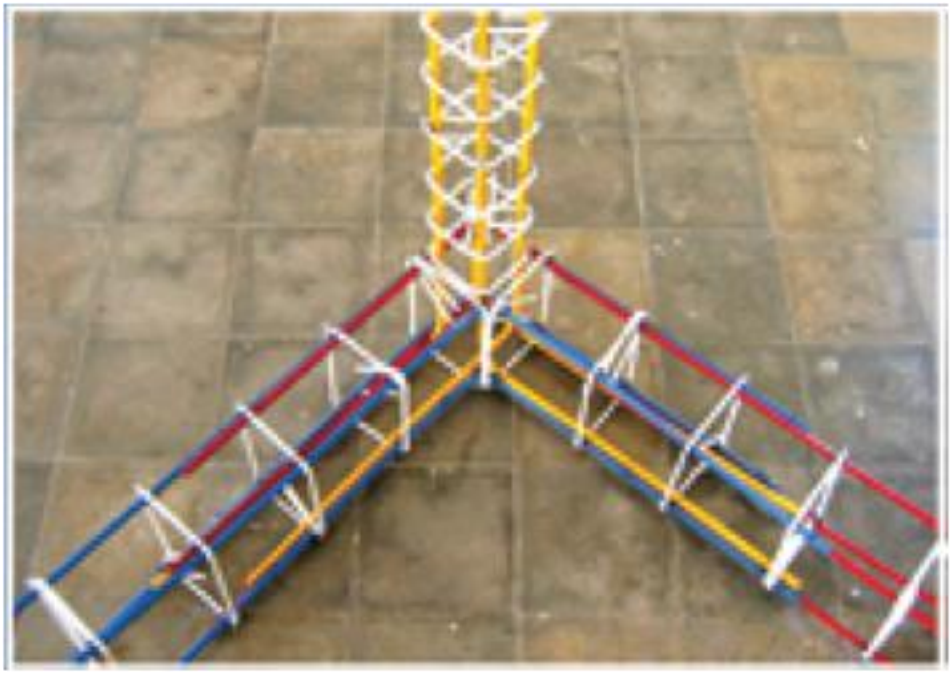


Figure A.15: Joint details of Beam and Column

Appendix B

DETAILS ESTIMATE

Appendix C

WALL DENSITY CALCULATION OF CONFINED METHOD

A 3-storied frame structured building was analyzed in confined method to check earthquake resistant capacity by a parameter wall density index, p_A or d. According to eurocode8, wall density index, $p_A \geq 3\%$ for 3-storied building located in moderate seismic zone, region having peak ground acceleration (PGA) equal to 0.15g that applicable to seismic zone-II in Bangladesh, Noor, Ansary, and Yasin (2005).

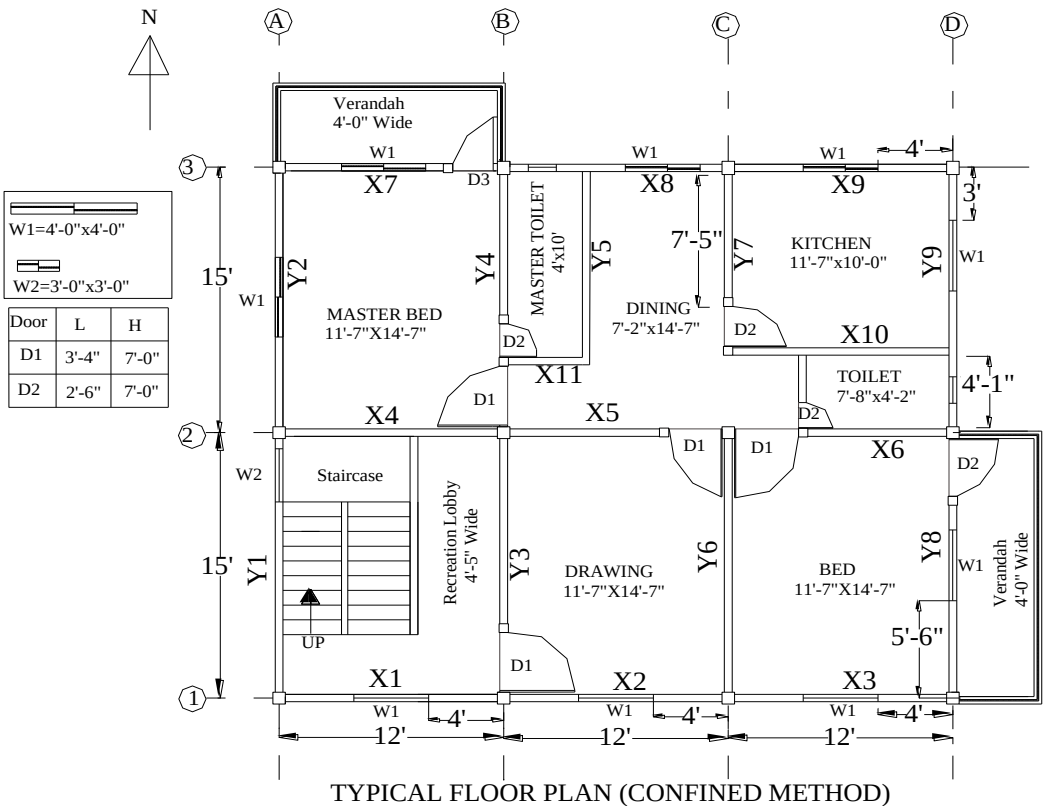


Figure C1: Typical Floor Plan of Confined Method

Wall was not considered in wall density calculation whose:

a) $H/L \geq 1.5$

b) Unconfined opening>10% of wall area.

Slenderness ratio, $H/L_X = 9.42/11.33 = 0.83 < 1.5$. So ok.

Slenderness ratio, $H/L_Y = 9.42/14.33 = 0.65 < 1.5$. So ok.

Window opening = 4'-0"x4'-0" = 16 sft.

Wall Areas:

X1=11'-4"x9'-5"=106.73 sft X2=11'-4"x9'-5"=106.73 sft

X3=11'-4"x9'-5"=106.73 sft X4=11'-4"x9'-5"=106.73 sft

X5=11'-4"x9'-5"=106.73 sft X6=11'-4"x9'-5"=106.73 sft

X7=11'-4"x9'-5"=106.73 sft X8=11'-4"x9'-5"=106.73 sft

X9=11'-4"x9'-5"=106.73 sft X10=11'-7"x9'-5"=109.08 sft

X11=4'-5"x9'-5"=41.64 sft

So opening area/wall area = 16/106.73 = 15% >10%. So window opening was deducted in wall density calculation.

∴ Length of wall in XX direction:

Grid 1-1 wall = 36'-8"-(4'-0"x3)	= 24'-8"
Grid 2-2 wall = 36'-8"-(3'-4"x2)	= 30'-0"
Grid 3-3 wall = 36'-8"-(3'-0"x4)-2'-6"	= 22'-2"
X10	= 11'-7"
X11	= 4'-5"
Total Length	= 92'-10"

So wall area, $A_{W,\,XX}$ = 92'-10"x0'-5" = 38.68 sft.

Floor area, A_P = 36'-8"x30'-8" = 1124.67 sft.

Wall density index in XX direction, $P_{A,\,XX} = (A_W/A_P)$ X 100 = (38.68/1124.67) x 100

= 3.44%>3%. So ok.

Area of Y1 = 14'-4"x9'-5" = 134.98 sft

Opening W2 = 3'-0"x3'-0"= 9 sft = 6.7% of Y1. So no deduction is required.

W2 in Y2 wall = 16/134.98 = 11.85%>10%. So W2 in Y2 wall was deducted.

Door area in Y3 wall = (3.33x7)/134.98 = 17.3%>10%. So door opening from Y3, Y4, Y8, Y9 walls has deducted.

Opening in Y7 wall = (2.5x7)/(10.33x9.42) =17.98%>10%. So D_2 from Y7 wall was deducted.

∴ Length of wall in YY direction:

Grid A-A wall= 30'-8"-4'-0"	= 26'-8"
Grid B-B wall= 30'-8"-(3'-4"x2)-2'-6"	= 21'-6"
Grid C-C wall= 30'-8"-4'-0"-2'-6"	= 24'-2"
Grid D-D wall= 30'-8"-(4'-0"x2)-2'-6"	= 20'-2"
Y5	= 11'-0"
	Total Length = 103'-6"

So wall area, $A_{W,\ YY}$ =103'-6"x0'-5"=43.47 sft.

Floor area, A_P=36'-8"x30'-8"=1124.67 sft.

Wall density index in YY direction, $P_{A,\ YY}$= (A_W/A_P) X 100= (43.47 /1124.67) x 100

=3.87%>3%. So ok.