SEISMIC RETROFITTING OF REINFORCED CONCRETE BUILDINGS

A DISSERTATION

Submitted in partial fulfilment of the requirements for the award of the degree

of

MASTER OF ENGINEERING

in

EARTHQUAKE ENGINEERING

(With Specialization in Structural Dynamics)

By

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CANDIDATE'S DECLARATION

I hereby declare that the work which is presented in this dissertation entitled **"SEISMIC RETROFITTING OF REINFORCED CONCRETE BUILDINGS"**, in partial fulfilment of the requirements for the award of degree of **MASTER OF ENGINEERING** in **EARTHQUAKE ENGINEERING** with specialization in **STRUCTURAL DYNAMICS**, submitted to the Department of Earthquake Engineering, University of Roorkee, Roorkee India, is an authentic record of my own work carried out for a period from August, 1999 to March, 2000 under the supervision of **Dr. S. K. THAKKAR**, Professor, Department of Earthquake Engineering, University of Roorkee, Roorkee.

The matter embodied in this dissertation has not been submitted for the award of any other degree.

Dated: 25/3/2000
Place: Roorkee

Amar prakash.
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This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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ABSTRACT

Recent earthquakes in many parts of the globe have revealed the issues regarding the vulnerability of existing buildings. The existing building structures, which were designed and constructed according to earlier code provisions, do not satisfy requirements of current seismic code and design practices. Many reinforced concrete buildings in urban regions lying in active seismic zones, may suffer moderate to severe damages during future ground motions. Therefore it is essential to mitigate unacceptable hazards to property and life of occupants, posed during future probable earthquake. The mitigation of hazards is possible by means of seismic retrofitting of inadequate existing building structures.

The present dissertation work deals with the seismic retrofitting of an existing fourteen storied reinforced concrete building located in seismic zone IV. The study comprises of seismic evaluation and retrofitting of R. C. building by using steel bracing and infill masonry walls. The seismic evaluation is carried out on the basis of two-dimensional elastic, linear and dynamic analysis. The exterior frames are studied for comparison of performance of methods of retrofitting. The computer software package STAAD III (1997) is used for dynamic analysis.

The study concludes with the fact that the building designed as per provisions of IS: 456-1978 using limit state method of design, and analyzed as per existing seismic code IS:1893-1984, is inadequate for provisions of revised draft code. The performance

of the existing inadequate buildings can be improved by adding steel bracing or infill masonry walls. The pattern of steel bracing, or location of infill masonry walls affect the response of the existing inadequate buildings.

Three patterns of steel bracing are used in the present study, in which, cross pattern, shows better performance than V and diamond pattern, but is relatively expensive. However, diamond pattern gives moderate performance as compared with V and cross pattern. The infill masonry wall Type 1, gives better performance than type 2, for the same amount of infill material. The infill masonry walls are relatively better and economic method of retrofitting, but they create more obstruction to light, cross ventilation and view, as compared to steel bracing method.

CONTENTS

LIST OF NOTATIONS

α_h = horizontal seismic coefficient

ω_n = natural frequency of the system

α_o = basic horizontal seismic coefficient

θ = angle between diagonal strut and the horizontal

ξ = damping ratio

μ = friction coefficient

Γ = mass participation factor

Φ = mode shape vector

τ = shear strength of masonry

λ = slenderness ratio

β = soil foundation factor

c = damping matrix

D = largest plan dimension of building

D.L. = dead loads

E.L. = earthquake loads

E_c = modulus of elasticity for concrete

E_m = modulus of elasticity for masonry

f_{ac} = allowable compressive strength for steel section

f_{at} = allowable tensile strength for steel section

f_{ck} = characteristic strength of concrete

f_m = compression strength of masonry

f_y = yield stress of steel reinforcement

H = height of building

h_e = effective column height between column hinges

h_m = height of infill

I_g = gross moment of inertia for concrete column

K = performance factor

k = stiffness matrix

L,l = length of members

L.L. = live loads

m = mass matrix

P(t) = dynamic force

R = seismic force reduction factor

R_c = diagonal compressive failure force

R_s = equivalent diagonal strut compression force

S_a/g = average response acceleration coefficient

S_{an} = spectral acceleration

S_{vn} = spectral velocity

t = thickness of infill

T = time period

V_B = design base shear

V_f = maximum shear force resisted by infill panel

w = effective width of diagonal strut

W	= seismic weight of the building
z	= vertical contact length between panel and column
Z	= seismic zone factor

W = seismic weight of the building

z = vertical contact length between panel and column

Z = seismic zone factor

LIST OF TABLES

LIST OF FIGURES

Safety of life in the seismic event is the prime consideration of earthquake resistant design philosophies. Experience from the past earthquakes has revealed that much loss of life and property results due to inadequacies and faulty practices in seismic design of structures. To improve the behavior of inadequate building and to minimize the damage it is essential to retrofit the inadequate buildings.

The seismic evaluation and retrofit of existing building pose a great challenge for the owner, architect, engineers and building officials. The risk measured both in lives and money, are high. Equally high is the inevitable uncertainty of Where, When and How large earthquake will be? The inherent complexity of reinforced concrete buildings and of their performance during earthquakes compounds the uncertainty. Traditional design and analysis procedures developed primarily for new constructions are not wholly adequate tools for meeting this challenge.

Therefore retrofitting is a challenging task because limited choices left to play with. Furthermore the insurance of desired strength level in retrofitted structure is not an easy task. A number of retrofitting techniques has been investigated now a day.

1.1 SEISMIC DESIGN

Earthquake phenomenon in nature is very complex one, involving many parameters in imparting the inertial forces developed in the masses of the structure. A proper understanding of the development of these forces is an uppermost criteria to arrive at a safe seismic design procedure. A sense of unpredictability is always associated with seismic activities as regards its size and degree of intensity. The general aim, therefore, of seismic design can be defined as providing adequate safety levels with respect to collapse during exceptionally intense earthquakes as well as with respect to damage to adjacent buildings.

LIST OF FIGURES

CHAPTER-1

INTRODUCTION

1.0 GENERAL

Seismic retrofitting is the process of upgrading the structural strength of an existing damaged or undamaged structure to make it capable of resisting future probable earthquake generated forces.

Earthquake disaster has always been one of the great natural calamities thrust upon the mankind since time immemorial and bringing in its wake untold miseries and hardships to the people affected. Indian subcontinent has been experienced with some of the most severe earthquakes in the world. The youngest mountain series of Himalayas covers whole northeast boundary regions of India. The tectonic activities are still continuing which may result into severe earthquake in future as anticipated by many scientists and researchers. The recent earthquakes in Indian subcontinent such as of Uttarkashi (October 20, 1991), Latur (September 30, 1993), Jabalpur (May 22, 1997), and Chamoli (March 30, 1999) have again reminded the society of concerned engineers and scientists to think about safety measures for earthquake protection. Therefore it is essential to seismically evaluate the many existing buildings as per current code requirements. The buildings found inadequate for resisting future probable earthquake needs to be retrofitted.

Many reinforced concrete moment resisting framed buildings lack strength and ductility. In order to correct these deficiencies various techniques have been developed. In the present study a fourteen storied-reinforced framed building have been retrofitted by using steel bracing in three patterns (V, diamond and cross) and infill walls.

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Further it aims to protect structures against excessive material damage under the action of moderate intensity earthquakes. Importance is given to safeguard the safety and the comfort of the occupants by limiting the structural response to a predefined tolerable limit. Panic caused due to the earthquake induced shaking among the occupants which can be hazardous, is another aspect to be covered in a good seismic design. Principles underlying the earthquake resistant design of buildings have always been to achieve the objectives stated above and striving for better understanding of the structural responses to the earthquake induced ground motions.

The capacity spectrum method is gaining acceptance in seismic design of multistoried building structures, which is actually performance-based design procedure. Two key elements of the performance-based design procedure are capacity and demand. Demand is the representation of the earthquake ground motion means forces induced in members due to ground motion. Capacity is a representation of the structure's ability to resist the seismic demand. The overall capacity of a structure depends on the strength and deformation capacities of the individual components of the structure.

Extensive research works are underway in all the earthquake prone countries namely Japan, U.S.A., New Zealand, India, China, Russia etc. among others.

1.2 NEED OF RETROFITTING

The retrofitting is one of the best options to make an existing inadequate building safe against future probable earthquake forces. Because it may not be a wise step to demolish the existing inadequate building, in spite of that strength can be added for a little additional cost. There are many other factors, considered in decision making for any retrofitting strategy.

The following are some reasons that may need retrofitting:

1. Building which are designed considering gravity loads only.
2. Development activities in the field of EQRD of buildings and other structures result into change in design concepts.
3. Timely revisions in codes of practice and standards.
4. Revisions in seismic zone map of country.
5. In cases of alterations in buildings in seismic prone area i.e. increase in number of story, increase in loading class etc.
6. In cases of deterioration of EQ forces resistant level of building e.g. decrease in strength of construction material due to decay, fire damage, and settlement of foundations.

1.3 OBJECTIVES OF STUDY

The present study has following objectives:

(i) To evaluate safety of the existing reinforced concrete building according to revised seismic code.

(ii) To carry out seismic analysis of building retrofitted by using (a) Steel bracing in different patterns, and (b) Infill walls at various locations.

(iii) To determine the relative merits and demerits of the steel bracing and infill wall method of retrofitting.

1.4 SCOPE OF STUDY

The present study is an attempt in the state of art of seismic retrofitting of reinforced concrete buildings. The focus of attention is to increase capacity demand ratio (C/D ratio) by creating minimum intervention and without much modification in interior planning of the building. The study is based on response spectrum analysis. Further the foundation effects and soil-structure interaction is not included, assuming that the foundation is safe as the raft foundation system is provided. Since the building is symmetric and regular in plan and elevation, therefore 2D analysis is carried out.

The analysis has been carried out using STAAD-III software package (1997).

1.5 ORGANIZATION OF DISSERTATION

This dissertation is organized in five chapters.

1. General introduction, seismic design concept, and need of retrofitting is covered in first chapter.
2. The second chapter presents a review of literature. The vulnerability of RC framed buildings, evaluation procedures, principle of retrofitting and techniques of retrofitting are briefly reviewed in this chapter.
3. Third chapter deals with seismic evaluation of 14-storied reinforced concrete framed building.
4. The chapter four comprises of seismic retrofitting of the existing 14-storied R.C. framed building using infill masonry wall and steel bracing (V, diamond, cross patterns), along with the results of analysis and discussion on relative merits and demerits.
5. The chapter five deals with summary and conclusions arrived at and scope for further study.
6. An appendix is given to show some sample calculations for design of cross pattern brace, and infill masonry wall.

CHAPTER-2

REVIEW OF LITERATURE

2.1 SEISMIC VULNERABILITY OF REINFORCED CONCRETE FRAMED BUILDINGS

The performance of a building under seismic event depends upon its inherent physical attributes such as strength, stiffness, ductility, damping and characteristics of the ground motion. The improper distribution of these attributes may drastically affect the seismic performance of the building.

2.1.1 MAIN ATTRIBUTES OF THE STUCTURES

The following attributes govern the dynamic response of all structures to ground motions. These attributes can be best achieved only in an idealized structure but it is quite difficult to achieve while retrofitting an existing structure. The attributes are so interlinked that any change in one will affect the other.

(i) Strength

One of the prime considerations in seismic retrofitting is the strength of existing building. Strength is the force resistance capability of materials used.

(ii) Stiffness

The distribution of forces among various members depends on their stiffness. Actually it is geometrical parameter and depends upon elastic modulii and dimensions of various members. It plays an important role in force transmission through members.

(iii) Load Path

A continuous and complete load path is essential for the proper seismic behaviour of a structure. Missing links in the load path must be identified load path evaluation should begin by establishing the source of all lateral loads generated in building and then tracking how those forces travel through structural system.

(iv) Vertical Irregularities

The vertical irregularities typically occur in a storey, which is significantly weaker, more flexible or heavier. These irregularities are normally due to significant changes in building configuration such as setbacks, discontinuous vertical elements or changes in storey heights. However they sometimes arise due to changes in column dimensions size and number of main reinforcing steel bars or column tie spacing.

(v) Horizontal Irregularities

The horizontal irregularities in RC framed buildings are typically due to odd plan shapes, re-entrant corners, or diaphragm etc. These irregularities often cause eccentricities resulting in torsion response to ground motions which further lead to concentrated demands on diaphragm and deflections at building ends.

(vi) Weak Column/ Strong Beam

Optimum seismic performance is gained when frame members have shear strengths greater then bending strength and when bending strength of column is greater than the beams. These features provide a controlled failure mode and in multistory buildings increase the total absorption capacity of system. In older concrete frame buildings where the beam are often stronger than the columns, column hinging can lead to a storey mechanism creating large P-Δ effects and inelastic rotations in the columns. This is undesirable because it may lead to loss of the column's gravity load carrying capacity after only a very few cycles.

(vii) Poor Detailing Of Reinforcement

The R C framed buildings in eighties and earlier in India and abroad contains an array of non-ductile detailing of reinforcement in beams, columns and joints. As a result exhibit poor seismic performance.

(viii) Frame Wall Interaction

In concrete frame wall system the relatively stiff wall that prevents the frames from experiencing large lateral displacements and deflections. The improper distributions of walls give rise to component and connection failure mechanisms such as diagonal compression, sliding shear etc.

(viii) Adjacent Building

Insufficient seismic gap between two buildings may result in hammering failure.

2.1.2 CAUSES OF FAILURE OF R. C. BUILDINGS

The awareness of causes of failure of reinforced concrete buildings is essential to tackle problems during retrofit. The main causes of failure of reinforced concrete buildings investigated in past earthquakes are as follows:

(a) Lack of good design and planning of components of buildings (beams, columns, joints, infill walls etc.).

(b) Poor quality of construction and workmanship.

(c) Inadequate detailing of reinforcement in various components particularly at joints, columns and beams from ductility considerations.

(d) Inadequate diaphragm action of roof or floors.

(e) Inadequate treatments of infill masonry wall that may not permit complete frame action.

(f) Upgraded seismic zones in the national standard may sometimes indicate earthquake forces higher than designed forces may be a cause.

The deficiencies in detailing of reinforcement noticed in various components in past earthquakes are as follows:

(i) IN BEAMS

- Inadequate detailing of splices hooks and stirrups.
- Longitudinal reinforcement in beams not adequately embedded in the joints.
- The reinforced ratio is not adequate to meet required ductility.

(ii) IN COLUMNS

- Low value of longitudinal reinforcement.
- Inadequate splices in longitudinal reinforcement.
- Larger pitch of transverse reinforcement in plastic hinge region columns.

(iii) IN JOINTS

- Little or no transverse reinforcement within the joint.
- Inadequate anchorage of beam reinforcement in joint.
- Sometimes built-up sections are used as reinforcement in case of high-rise buildings where the welding is used to connect beams and columns therefore faulty welding may also cause joint failure.

2.2 SEISMIC EVALUATION OF REINFORCED CONCRETE FRAMED BUILDINGS

The seismic evaluation and retrofitting of existing RC buildings pose a challenge for the owners, architects, engineers and building officials. The essence of virtually all seismic evaluation procedures is a comparison between some measure of the demand that probable earthquakes place on a structure to a measure of the capacity of the building to resist. Seismic evaluation includes:

(i) Visual inspection and collection of general information.

(ii) Experimental procedures for modified material strength and safety factors.

(iii) Performance objective and modified seismic action.

In India there is no document concerning to seismic evaluation, published so far. In USA various organizations provided comprehensive documents for seismic evaluation. Recently, FEMA-178 (BSSC 1992) developed a methodology for evaluation of seismic vulnerability of individual buildings. Gaining experience from performance of FEMA-178, handbook for seismic evaluation of building has been developed i.e. FEMA310. The applied technology council report (ATC-40, CSSC-1996) provides systematic guidance on how to investigate RC buildings for subjected to ground motions. In this document pushover analysis and performance objective have been discussed in detail. In Japan a number of buildings have been evaluated on the basis of judgement and performance indices. Simple, straight forward guidance to engineers on the advantages and disadvantages of various strengthening strategies and direction to follow have been presented by Loring, A. and Wyllie, Jr., (1996). A comprehensive paper (Thakkar, 1994) on repair, seismic strengthening and retrofitting of existing, damaged and undamaged buildings presents general aspects in state of art of seismic retrofitting.

Finally, to evaluate strength of existing or damaged building for which suitable mathematical models required to be made. A detailed seismic analysis linear or non-linear, using any of response spectrum method or equivalent lateral force method or capacity spectrum method (i.e. pushover analysis) would be necessary with computer software. The capacity demand estimation for members then can be done to determine capacity demand (C/D) ratio for members. If C/D ratio less than one, the retrofitting is needed.

2.2.1 MODIFICATIONS MADE IN EXISTING CODE IS: 1893-1984

Following are the major modifications made in existing code IS: 1893-1984 and are presented in revised draft code.

(a) The seismic zoning map has been revised which divided the India into four seismic zones instead of five as in the existing code IS: 1893-1984. The seismic zoning is shown in Fig (2.1). The seismic zone factors have been modified. The modified values show more realistic values of effective peak ground acceleration for maximum credible earthquake (MCE) for zones.

(b) Revised draft provides response spectra for two types of site i. e. Rock type and soil type, at 5% critical damping ratio. The multiplying factors are given for other damping ratios.

(c) The expression for estimating fundamental time period of multistoried buildings has been revised.

(d) The revised draft follows the procedures such as first calculate the actual forces experienced by structure for MCE in a specified zone. Then reduction in response is done by introducing the response reduction factor (R), which depends upon type of construction in place of performance factor in earlier code.

(e) A lower bound is specified for the design base shear (V_B) of the structure based on empirical relation of the of fundamental natural time period, T and that calculated by dynamic analysis (V_B).

(f) The soil-foundation factor (β) has dropped and a clause to restrict the use of foundations vulnerable to differential settlement in severe seismic zones is introduced in draft code.

(g) For buildings with irregular plans the torsional eccentricity values have been modified on higher side.

(h) Modal combination rules for dynamic analysis has been revised.

The salient features of revised draft and IS:1893-1984 have been briefly mentioned in Table 2.1.

2.3 GENERAL PRINCIPLES OF RETROFITTING

The underlying concepts and principles of strengthening of buildings are enumerated below:

(a) Before adopting any retrofitting scheme the structural inadequacies of existing building should be seismically evaluated.

(b) All members and components of retrofitted structures must be suitably tied together so that integral action may be achieved during motion.

(c) Adequate structural members, strong and ductile connections between diaphragm walls and foundations should be ensured.

(d) High quality of construction and use of special binding material is key to retrofitting.

(e) Addition of new elements like shear walls, infill walls etc. should ensure adequate connection between old and new construction using shear keys and suitably designed dowels.

(f) Besides strengthening and increasing ductility reduction in dead load may have to be recommended in some cases to improve seismic performance e.g. reduction in number of stories.

2.4 MATERIALS USED FOR RETROFITTING

Since retrofitting work deals with connections of new material to the existing building members therefore material should provide good bond and anchorage to get desired performance level. Bureau of standards of USA (ATC-3, 1979) has suggested a number of materials for the purpose of repairing and retrofitting. Commonly used materials are as polymer concrete, latex modified concrete, fiber reinforced concrete, gypsum cement concrete, mechanical anchors, shotcrete, and slurry infiltrated mat concrete (Simcon) etc.

In this study the steel sections having yield strength 250 N/mm^2 are used for bracing and brick masonry having elastic modulus 14400 MPa is used for infill wall with 1:2 (cement: sand) mortar.

2.5 RETROFITTING TECHNIQUES

The structural engineers are accustomed with the design of new reinforced concrete buildings to achieve desired levels of safety by incorporating features for required strength and ductility. There are codes available for design of RC buildings but there are little or no guidelines available for strengthening of existing buildings. The conventional codes and methods may not be applicable to retrofitting problems. There are number of innovative ideas developed in last decades for retrofitting of existing reinforced concrete buildings some of the methods employed in the state of art on the subject are described below:

2.5.1 ADDITION OF NEW SHEAR WALLS

The introduction of new shear walls from base to roof in the existing RC framed buildings is one of the most commonly used approaches to seismic retrofitting. Shear walls add strength and stiffness to the buildings. These walls should be tied to the existing structure. The locations of the shear walls should be so chosen as to minimize the eccentricity.

2.5.2 ADDITION OF INFILL WALLS

The infill walls of R.C., masonry or precast concrete elements can be added to frame buildings. This method can increase the strength of the inadequate buildings appreciably. The precast infill panels have been used to strengthen the non-ductile R.C. framed buildings (Frosch, Kreger and Jirsa 1996).

In the past a lot of retrofitting work have been done using infilled frames. The selection of infill material depends upon the magnitude of forces, availability and strength of infill material. The studies made by researchers (Bertero and Brokken, 1981) have concluded with the fact that use of solid bricks used externally with welded wire fabric covered with cement mortar and anchored to the frame provides best performance among clay brick, concrete (light weight), hollow and solid bricks and concrete blocks.

The previous investigations done by Klinger and Bertero, 1976 have shown that the infill affects significantly the stiffness, strength and deformation capacity (i.e. ductility and energy dissipation and absorption capacities) of the bare frames. All these effects result in changes in dynamic characteristics of the building in which the infill is used. The changes depends upon how the infill is constructed and integrated (i.e. anchored or connected) to the bare framed building. The addition of infills brings an increase in building mass, which results, into increase in reactive mass and time period. By virtue of infill wall addition, increase in stiffness takes place, which decreases the time period 'T'. Therefore, these opposite and interactive effects as well as the changes in the mechanism of dissipation of energy by inelastic deformations make it difficult to arrive at definite conclusions regarding the final effects of the infill. To achieve desired results with infill walls the quality control and workmanship particularly in the attachment (anchorage) of panels to the frame should be emphasized.

2.5.3 ADDITION OF STEEL BRACED FRAMES

The steel bracing can be added to existing inadequate building frames to increase the strength and stiffness of building without much addition of mass. Typically braced frames provide lower level of stiffness and strength than do shear walls. Bracing patterns can be adopted as per the aesthetic and strength point of views. Various kind of bracing patterns are used such as single diagonal, latticed, k, knee etc. Hiroyaqsu, Usami et.al. (1988) has strengthened three storied framed buildings in Japan. The use of ductile steel bracing for strengthening existing seismically weak R.C. slab column building structure has been studied (Goel and Masri, 1996). An analytical study (Bouadi and Engelhardt, 1996) has carried out in which the use of the steel eccentrically braced frames (EBFs) for retrofit of the existing reinforced concrete building was investigated.

Though the use of cross bracing is more common in steel structures of industrial and commercial buildings and transmission towers two resist horizontal wind and seismic loads. Recently many R.C. framed buildings in Japan and USA have been designed and retrofitted for seismic forces using steel braced frames. Because of the ease in handling steel sections further light in weight advocates the use of rolled steel sections such as angles, pipe, channel etc. The steel brace create less obstruction to light and air. In the present study the double angle braces are used in three patterns (V, diamond and cross) to study their performance while used in retrofitting.

The cross bracing derives its primary benefit from the interconnection of two diagonals, which cuts down their unsupported buckling length in compression. Recently several studies have focussed attention on the out of plane buckling of cross bracing as designed factors of considerable importance. Most of these study (De wolf and Pelliccione, 1979; Kitipornchai and Finch, 1986; Picard and Beaulieu, 1987; Stoman 1988, 1989; Wang and Boresi, 1992).

The design of compression braces is usually based on the recommended effective length equal to diagonal length (De wolf and Pellccione, 1979), to half the diagonal length (Picard and Beaulieu, 1987) or 0.85 times half the diagonal length (EL-Tayem and Goel, 1986) for a simply supported cross bracing.

2.5.4 JACKETING OF REINFORCED MEMBERS

The deformation capacity of non-ductile concrete columns can be enhanced through provisions of exterior confinement jacketing. Jacketing can be done from one or more sides of the members. Numerous buildings have been retrofitted by this procedure (Guerrero, Gomez and Gonzalez, 1996). Steel jackets for seismic strengthening of columns have also been employed. This method is useful when columns have inadequate lap splices in longitudinal reinforcement and inadequate transverse reinforcement in columns (Aboutaha and Jirsa, 1996).

Corrugated steel jacketing has also been proposed for retrofitting of beam column joints (Ghobarah, Aziz and Biddah, 1997) the corrugated steel jacketing provides better lateral confinement as compared to rectangular flat jackets.

Carbon fiber binding method is a new seismic retrofit method for the existing RC columns. The method is considered superior to steel plate jacketing or RC jacketing. The structural performance, evaluation of shear capacity and ductility has been well-studied (Katsumata and Kobatake, 1996).

Jacketing with SIMCON (Slurry Infiltrated Mat Concrete) is a variety of high performance fiber RC (HPFRCs). The premade fiber mats are wrapped around the existing concrete elements. Thus cracking in plastic hinge regions is delayed giving gradual failure of members and the strength increases too (Ajiboye, F.O.et.al., 1998). SIMCON has been investigated and documented by Oluokun and Malak et.al.

2.5.5 SUPPLEMENTAL DAMPING DEVICES

The supplemental damping devices have been used by (Miyamoto, and Scholl, 1996). Another way of providing supplemental damping to inadequate building is by incorporating simple and inexpensive friction damping devices at strategic locations in buildings. Their earthquake resistance and damage control potential is dramatically increased. The friction dampers are simple and foolproof in construction, possess very high-energy dissipation capacity and provides reliable and maintenance free performance over the life of building. Another friction damping device is introduced (Nims, 1993) whose main features is reported to be a self centering capability. Canadian space agency building have been retrofitted with friction dampers (Pall et. al. 1993).

2.5.6 SEISMIC BASE ISOLATION TECHNIQUE

This approach requires the insertion of complaint bearings within a single level of building's vertical load carrying system, typically near its base. The bearings are designed to have low stiffness, extensive lateral deformation capacity and may have superior energy dissipation characteristics. Installation of an isolation system results in a substantial increase in the building's fundamental response period and potentially its effective damping. Base isolation has most commonly been used in the past as a method of retrofitting historic structures. The significant reduction in displacement response and acceleration that occur within the super structure of an isolated building results in much better performance of equipment, systems and other non-structural elements than is attainable with most other retrofit systems.

2.6 OTHER TECHNIQUES OF RETROFITTING OF R.C.C. MEMBERS

Warner [42] has stated that, very often a need arise to retrofit some of the members e.g. beams, columns, foundations etc., to resist increased forces due to earthquake ground motion. Such retrofitting and modifications are particularly difficult for R.C.C. members (i.e. beams and columns) due to monolithic construction and hidden reinforcement. The following are the methods for retrofitting of individual members:

2.6.1 COLUMN STRENGTHENING

Warner [42] and Karamchandani [21] have suggested the following methods for retrofitting of existing R.C. columns depending upon availability of equipments and ease of construction:

(a) **Propping Up** – One of the simplest and most effective method for retrofitting a column in an existing building is to partially unload the column by jacketing between floors and then insert two or more props to carry portion of axial load. The props are usually rolled steel sections, which may subsequently be incased in concrete to improve fire protection and appearance. The disadvantage of this procedure is that considerable floor space is lost and that props may not be effective in transferring moments unless positive connections are introduced at both ends e.g. in the form of end plates bolted through holes drilled in the floors.

(b) **Sleeving** – The sleeve consists of additional longitudinal and horizontal steel ties on ram setted to the existing column and with a cast in place concrete and gunite cover.

(c) **Collar type** – A simpler, cheaper but aesthetically less pleasing means of providing moment transfer is to introduce a steel collar, which fits snugly to the column end to under side of the upper floor the steel collar may be either bolted or glued.

(d) **Casing** – In casing additional reinforcement and concrete is added on four sides of the concerned member. This method is suitable for interior columns.

(e) **Jacketing** – A jacket is a reinforced concrete layer on three sides of the concerned member. This method is suitable when one face of column is flushed to wall and the other three sides are open.

(f) **Building up** – The building may be used on one or two faces of the column.

During the retrofitting process the column should be partially unloaded so that the total axial load is finally shared between the new and old construction material. The thickness of the wall of the casing or jacket or build-up depends upon the amount of reinforcement, the size of protective layer and the method of application of layer.

The effectiveness of strengthening with the help of casing or jacket or build-up depends upon the degree of adhesion of the old and new concrete, which in turn depends on the condition of application of concrete mix, method of its compaction and finish of the surface to be connected. Due to contraction during setting and hardening, the reinforced concrete casing or jacket compress tightly the member to be retrofitted and works with it as an integrated whole.

2.6.2 STRENGTHENING OF SLABS

Though the floor system in an R.C. building is not a part of the structural systems, some times the need may arise to strengthen the weak slabs depending upon the type of weakness. The following methods can be used to strengthen the existing slabs.

(i) Underlay

This method is used to increase the sagging moment resisting capacity of slabs. Additional tensile reinforcement is fixed by ramjets to the under side of the existing concrete slabs and a concrete coating of proper thickness is applied to cover the new reinforcements.

(ii) Overlay

This method is used for increasing hogging moment carrying capacity. A new concrete layer with additional tensile reinforcement is placed over existing slab.

2.7 SEQUENCE OF RETROFITTING WORK

Karamchandani has given some guidelines for sequence of retrofitting work. From safety of structure considerations, proper and safe execution of retrofitting works and ensuring active participation of the new concrete and reinforcement with the old one, the member is relieved of the load upto extent possible.

After the member/structure is relieved of the load the following sequential steps are adopted:

(1) Protective cover of the concrete is removed at the required places and reinforcement exposed by minimum half the diameter, the rust and concrete cleaned off.

(2) The new reinforcement and inserts are placed in position and welded to the reinforcement exposed.

(3) All contact surfaces are cleaned off dust by compressed air and washed with water.

(4) Shuttering is installed.

(5) Concrete is poured into forms.

The usual practice is to use Portland cement based fine-grained concrete of plastic consistency with slump value 8-10 cm. The maximum size of coarse aggregate used is limited to one-third of the minimum thickness of the concrete layer on the surface or three-fourth (3/4) of the minimum bar spacing, whichever is minimum.

CHAPTER-3

SEISMIC EVALUATION OF 14-STORIED R. C. FRAMED BUILDING

3.0 GENERAL

In this chapter the seismic evaluation of 14-storied reinforced concrete building in seismic zone IV, is carried out on the basis of results of dynamic analysis. The capacity demand (C/D) ratios are determined for members of frames of existing building. The capacities of various sections are determined using limit state method following the IS: 456-1978 code. The demands are obtained from the dynamic analysis using revised draft code spectra and site spectra (i.e. Response spectra used for Cycle gas based power project site, Faridabad project report EQ 95-17). The design basis earthquake is considered for elastic linear dynamic analysis. Later on the safety of existing building is assessed on the basis of the C/D ratio. The member showing C/D ratio less than 1, represent distress and need to be retrofitted.

3.1 DESCRIPTION OF BUILDING

The object building is located in New Delhi that falls in seismic zone IV of seismic zonal map of India as per revised draft code, Fig. 2.1. The building is a hospital having 14-stories (G+13).

The building consists of an assembly of cast in place reinforced concrete beams and columns. The typical plan of existing building is shown in Fig. 3.1. The building consists of five R.C. framed blocks (i.e. Block B_1, B_2, B_3, B_4 and central block). The central block have all the staircases and elevators. The frames of block B_2 are considered in the study. The typical plan of block B_2 with global directions used in analysis is given in Fig. 3.2(a). The Fig. 3.2(b) shows the strategic locations of the retrofitting methods in plan. A three dimensional view of existing block B_2 is shown in Fig. 3.3. The three dimensional view shows that the frames are having spandrel walls (300 mm thick and 1100mm high) in between columns, above these walls the glazed windows are provided. The first bay in z-direction has corridor openings etc. Concrete ordinary moment resisting frames (OMRFs) are expected to resist the lateral forces induced by earthquakes in existing building. The frames in x-direction consists of 4 bays @ 6.5 m each and 14 stories @ 3.1 m (i.e. floor to floor) each. The frames in z-direction direction have 3 bays @ 6 m and 14 stories @ 3.1 m each. The Fig. 3.4 shows typical frames with joint numbering. The joint number has four digits the first digit shows the column number of the storey, middle two digits shows the floor level and the fourth digit that is 1, represent that is joint of R. C. beams and columns. The member numbering is shown in Fig. 3.5. The R. C. frame member number has four digits, in which the first place digit indicates beam or column number in a story, middle two digits represent storey number (i.e. 00 for ground floor and 13 for roof) the digit at fourth place from right side designate the beam (if it equals to 2) or column (if it equals to 1).

The supports of all the frames are considered as fixed with respect to translation and rotation. The raft foundation system is provided under the block B2, the effect of foundation system is not considered in the study.

The floors and roof of the building have 200 mm thick, monolithically cast reinforced concrete slabs, with beams and columns. The sizes of all beams of frames are

500x500 mm and those of all the columns are 700x700 mm. The detailed interior planning of building block is not available. The grade of concrete used in columns upto fifth floor level is M25 and in above floors M20 grade concrete is used. The grade of concrete for all beams and floor slabs is M15. The grade of steel used for all columns, beams and floors is Fe 415. These details are also given in Table 3.0.

3.2 SEISMIC EVALUATION METHOD

The seismic evaluation of 14-storied R. C. building is carried out for the proposed revision in existing seismic code IS: 1893-1984. The details of the reinforcements in members are not given in available drawings hence determined on the basis of given gross sectional and material properties. The details of reinforcement in ground floor beams and columns are given in Fig. 3.6 and for ground floor beam-column connection are shown in Fig. 3.7.

The capacities of the frame members have been determined using limit state design method following the IS: 456-1978 code of practice. The C/D ratios for bending moments in beams are determined at support and mid span sections. These sections are denoted as I (i.e. section at the start), J (i.e. section at the end) and section at middle of the span. The capacity and demand for member shear forces are determined at the end sections of beams and denoted as I (for section at the starting joint) and J (for section at the end joint). In the columns the capacity and demands for bending moments and shear forces are determined at section I (i.e. near bottom joint of column) and section J (i.e. near top joint of column). The maximum responses among all load cases are used for obtaining the demands in members.

The original building block as described in section 3.1 is found regular in plan and elevation. It has no plan irregularity like re-entrant corners, diaphragm discontinuity, out of plane offsets. The variation in grade of concrete used in columns is not expected to introduce any vertical irregularity. The continuous load paths between the foundation, all diaphragm levels, beams and columns is available. But the lateral force resisting frames (OMRFs) are found weaker to resist design basis earthquake forces determined as per revised draft code.

3.3 ASSUMPTIONS IN DYNAMIC ANALYSIS

The following assumptions are made in dynamic analysis:

1) In 2D analysis plane frame behavior is considered for the building.
2) The bases of the frames are assumed as fixed with respect to rotational and translational movements.
3) Soil structure interaction effects are neglected.
4) The ground motion due to earthquake is applied in the horizontal direction of the frame. The effect of motion in vertical direction is neglected.
5) Bare frames are analyzed for incoming loads from slabs, partitions, and selfweights of the members and infill effect is not considered in analysis.
6) The contribution of the floor slabs to the flexural capacity of the beams is neglected as the slab in tension side may get fine crack. This assumption may give conservative results.
7) The criteria for considering number of modes is that more than 90% of mass should participate.

8) The damping of the structure is assumed to be constant as 5%, actually damping also changes with strength and stiffness.

9) The torsion effects are neglected on account of regular and symmetric shape of building.

10) The overturning and P-delta effects are neglected.

3.4 LOAD COMBINATIONS

In the analysis for capacity and demand the following load combinations conforming to revised draft code are used:

(a) 1.5 D.L. ± 1.5 L.L.

(b) 1.2 D.L. ± 1.2 L.L. ±1.2 E.L.

(c) 0.9 D.L. ± 1.5E.L.

(d) 1.5 D.L. ± 1.5 E.L.

3.5 MASS MODELLING FOR DYNAMIC ANALYSIS

The distribution of weights for gravity loading and mass modeling for dynamic loading is carried out as given below:

For gravity loads, the floor weights of respective tributary areas are distributed over beams after converting the triangular and trapezoidal loading into equivalent UDL. The weight of partition walls is included in the floor load intensities as 1.5 KN/Sqm.

For response spectrum method the masses are lumped only at the joints. 25% of live loads is considered to act with dead loads. The mass of spandrel walls of 1.1M height and 300 MM thick masonry is included in dead loads.

3.6 METHOD OF ANALYSIS USED FOR SEISMIC EVALUATION

There are two methods of dynamic analysis available i.e. time history analysis and response spectrum analysis. The time history analysis is truly dynamic in the sense that it determines the responses of the structure to a known ground motion at predetermined time steps. While on the other hand the response spectrum analysis consists in determining responses for few modes of vibration and then combining total response by suitable combination rules. Since the peaks of maximum response don't occur at same time, therefore modal combination rules such as square root of sum of squares (SRSS), or complete quadratic combination (CQC) are used to get peak response. The SRSS gives better results for structures having well spaced frequencies while CQC provides better results for systems having closely spaced frequencies. In this study the SRSS rule is used in this study because the frames have well spaced frequencies.

The response spectrum method is a dynamic analysis, because it uses the dynamic characteristics such as natural frequencies, natural mode and modal damping ratio of the structure and dynamic characteristics of ground motion through its response spectrum. The response spectrum method of dynamic analysis is use in this study, which is described in following section.

3.7 RESPONSE SPECTRUM METHOD

A brief explanation of the response spectrum method of analysis (Dowrick,1994) is given below:

The equation of dynamic equilibrium of multi degree of freedom system is:

$$m\ddot{x} + c\dot{x} + kx = P(t) \tag{3.1}$$

Where m, c and k represents mass, damping and stiffness matrices respectively. P(t) denotes the dynamic force. Dynamic response of a structure depends upon its frequency and displaced shapes the first step in dynamic analysis is therefore to find dynamic characteristics of the structure in free vibration damping is considered as zero. Considering the fact that in free vibration the motion is simple harmonic, equation (3.1) gives eigen value equation as:

$$k\hat{x} - \omega^2 m\hat{x} = 0 \tag{3.2}$$

Where, x is amplitude of motion. Solving this equation (3.2) for a building structure having N degrees of freedom vibration frequencies ω_n corresponding to mode shape vector ϕ_n. This vector ϕ_n represents relative amplitudes of motion for each displacement components in the mode n. Each mode of MDOF system behaves as a single degree freedom system (SDOF). Using orthogonality properties of mode shapes, equation (3.1) for MDOF system can be written as below:

$$\ddot{y}_n + 2\xi_n \omega_n \dot{y}_n + \omega_n^2 y_n = \frac{\phi_n^T P(t)}{\phi_n^T m \phi_n} \tag{3.3}$$

Where, y_n is generalized displacement in n^{th} mode and ϕ_n^T is row of mode shape vector corresponding to the column vector ϕ_n.

In terms of excitation by ground motion $x_g(t)$, the equation (3.3) can be expressed as:

$$\ddot{y}_n + 2\xi_n \omega_n \dot{y}_n + \omega_n^2 y_n = \frac{\Gamma_n}{\phi_n^T m \phi_n} \tag{3.4}$$

The factor Γ_n is mode participation factor, which is equal to $\phi_n^T m$ I where, I is a unit column vector of dimension equal to degree of freedom (N). Solving the above equation for y_n gives the response of the mode n at any time t. For solving the equation (3.4) Duhamel's integral can be used which gives following expression:

$$y_n(t) = \left(\frac{\Gamma_n}{\phi_n^T m \phi_n} \right) \cdot \frac{1}{\omega_n} \cdot \int_0^t x_g(\tau).e^{-\xi_n \omega_n (t-\tau)}.\sin\omega_n (t-\tau).d\tau \qquad (3.5)$$

If $y_{n,max}$ is maximum response in the n^{th} mode determined from equation (3.5) then the distribution of maximum displacement in the n^{th} is

$$\begin{aligned} x_{n,\max} &= \phi_n y_{n,\max} \\ x_{n,\max} &= \phi_n . \frac{\Gamma_n}{\phi_n^T m \phi_n} . \frac{S_{vn}}{\omega_n} \end{aligned} \qquad (3.6)$$

And the distribution of maximum earthquake forces in that mode is expressed as

$$\begin{aligned} P_{n,\max} &= m\phi_n \omega_n^2 y_{n,\max} \\ P_{n,\max} &= m\phi_n \frac{\Gamma_n}{\phi_n^2 m \phi_n} . S_{an} \end{aligned} \qquad (3.7)$$

Where, S_{vn} and S_{an} are the spectral velocity and spectral acceleration respectively.

Therefore in response spectrum method instead of considering the whole response time history of modes only peak response of mode is considered. The maximum response for each mode is computed from equation (3.5) and corresponding maximum displacement and force in each mode is determined using equation (3.6) and (3.7) respectively. Actually these maximum responses have already been included in response spectra curves for various values of ξ. Fig (2.2) shows the response spectrums for existing code, site and revised draft for soil site having damping 5% of critical. The undamped natural periods are determined from equation (3.2) and then maximum response corresponding to acceleration response read from response spectrums is determined.

3.8 DESIGN BASE SHEAR

The design base shear shears, V_B as per existing code IS: 1893-1984 and revised draft code were determined using following coefficients:

(a) For existing code spectra and site spectra

β=1.0, I=1.5, F_0=0.25, S_a/g as per response spectra given in Fig. 2.2.

(b) For revised draft code spectra

Z=0.24, I=1.5, R=3.0, S_a/g as per response spectra given in Fig. 2.2.

Damping is assumed 5% for all the response spectra.

3. 9 DISCUSSION OF ANALYSIS RESULTS

The results obtained from linear, elastic and two-dimensional dynamic analysis of frames in block B_2 are discussed in the following sections.

3.9.1 Time Periods and Mass Participation

The time periods obtained from dynamic analysis of frames in x and z-directions for first 10 modes are given in Table3.1. The fundamental time periods for edge frames in x-direction is 1.8018 sec but the empirical relationships of revised draft code for bare frames gives fundamental time period as 1.268 sec. For edge frame in z-direction the fundamental time period obtained from dynamic analysis is 1.7452 sec. and as per empirical relation of draft code is 1.268 sec. Therefore, the frames represent more flexibility as per dynamic analysis this may be because of not considering the effect of infill effects.

The mass participation for first mode is 78.87% and 78.65% in x and z-direction respectively. This shows that first mode has significant contribution to base shear forces. The total mass participation for all the ten modes considered are 99.43% for frames in x-direction and 99.44% in z-direction. The damping for all modes is taken as 5%.

3.9.2 Bending Moments

The bending moment capacity demand ratios for beams of edge frames in x-direction are given in Table 3.2. Few members at the top stories show the capacity demand ratio greater than one, these are shown in bold figures in the table. The majority of members have C/D≤1 at supports and mid span. The B.M. at node I increases upto 48.58% (i.e. corresponding to minimum C/D ratio which is 0.673). At middle section maximum increase in B.M. is 71.52%. Therefore the members are unsafe both at support and mid span.

The comparison between B.M. demands using revised draft code spectra and site spectra are given in Table 3.3 in which the demands as per revised draft code spectra (for soil site) are higher than the site spectra. Therefore the site spectra is not used in the study for determination of demands. Table 3.4 shows the B. M. C/D ratio for middle frame members in x-direction, which shows the C/D ratios, are less than unity. The few safe C/D>1 are shown in bold figures. The Table 3.5 gives the almost similar results for B. M. C/D ratio in frames at edge in z-direction.

The bending moment for the columns at bottom (i.e. node I) in edge frames in x-direction are given in Table 4.3 which shows an increase of 6.4% in ground floor (G/F) column. The C/D ratio for B.M. in frames at edge in z-direction is given in Table 4.8 which show the columns in G/F and first floor are unsafe for B.M.

3.9.3 Shear Forces

The shear forces in beams of edge frames in z-direction are given in Table 3.7, which show that the beams are safe at the end nodes I and J. The capacity demand ratios for shear forces in columns of frames at edge in x-direction are given in Table 3.6. In this table the demands are determined as per dynamic analysis, on the basis of ultimate B.M. capacities of beams and from short column (i.e. due to presence of spandrel walls) failure considerations. The demands as per short column failure are more than that of other considerations. The C/D ratios represent that the short column failure can occur in columns upto fifth floor and the columns above are safe for shear forces.

The base shears for existing frame in x and z-direction are given in Table 4.1(b) and 4.2(b) respectively. The factored storey shear forces for existing frames in x and z-direction are given in Table 4.15.

3.9.4 Joint Displacements

The joint displacements at the outer columns of edge frames in x and z-direction are given in Table 4.10 and 4.11 these are also plotted in Fig. 4.9 and 4.10 respectively. The value of joint displacements at roof level of building for edge frame is 7.1285 cm and 6.913 cm in x and z-direction respectively. The joint displacements for middle frame in z-direction are plotted in Fig. 4.11. The joint displacement at the roof level for middle frame in z-direction is 8.9908 cm.

3.9.5 Story Drifts

The permissible interstorey drift is limited to 0.004 times the storey height so that minimum damage would take place during earthquake and posing less psychological fear in the minds of people. The story drifts in frame at edge in x-direction are given in Table 4.12 and also plotted in Fig 4.12 along with story drifts obtained for retrofitted frame. For frames at edge and middle in z-direction storey drifts are given in Table 4.13 and are plotted in Fig.4.13 and 4.14 respectively. Though the drifts in lower floors are larger as compared with upper floors, but these are within permissible limits (i.e. 1.24cm).

3.9.6 Axial force

The columns of existing framed building are found safe for the demands. Because the minimum longitudinal reinforcement determined on the basis of IS:456-1978 is found adequate to resist the demands due to revised draft spectra. The axial load carrying capacities and demands as per revised draft code of the columns of the frame in x-direction are given in Table 4.5.. It can be seen that the demands due to revised draft are less than the capacities. Therefore the column at ground floor and above are found safe in the x-direction as given in table 4.3.1 also. Axial force demands due to revised draft in beams in z-direction as given in Table 4.14 are less than the capacities given in Table 4.9.

3.10 CONCLUSIONS

The building, which is evaluated for design basis earthquake forces as per, revised draft code is found inadequate. In the majority of beams in edge and middle frames are found weak because C/D ratios were less than one. The C/D ratios for B.Ms. at supports and mid span in beams upto eleventh floor are found less than 1. Only few beams in top floors are having C/D ratio greater than one and considered safe. The columns in edge frames at ground floor and first floor levels are found unsafe as B. M. demands are more than capacity. The possibility of short column failure due to spandrel walls, in columns upto fifth floor is also detected. The storey drifts for existing frames are found within permissible limits as per revised draft provisions (i.e. 0.004 times story height). Since ductile detailing code has not been followed in original design therefore frames are considered as ordinary moment resisting frames. Since majority of frame members (i.e. beams as well as columns) show C/D ratio <1, therefore retrofitting is needed. The methods used in seismic retrofitting and results obtained from dynamic analysis of retrofitted structures are described in next chapter.

CHAPTER-4

SEISMIC RETROFITTING OF 14-STORIED R. C. FRAMED BUILDING

4.0 GENERAL

On the basis of seismic evaluation carried out in the previous chapter it is found that building block B_2 need retrofitting to resist the earthquake forces. The seismic retrofitting can be done in various ways e.g. by strengthening of individual member or by adding lateral force resisting frame like shear walls, steel bracing, infill walls etc. Since the study building is a hospital structure therefore it is essential to maintain the health facilities during and after the earthquake. The strategy adopted in this study for seismic retrofitting is to strengthen the frames without creating much alteration and intervention to interior planning of building. Therefore the exterior frames are chosen to apply the retrofit methods and study the performance. In z-direction one middle frame is also used in addition to frames at edge. The results of dynamic analysis for retrofitted frames are discussed in this chapter. Finally their relative merits and demerits of the methods used in seismic retrofitting are described on the basis of the performance and quantities of material required in construction.

4.1 METHOD OF SEISMIC RETROFITTING

In the present study, the efforts are made to retrofit existing building by strengthening the entire frame instead of individual member strengthening. Two methods are used (a) adding steel bracing and (b) adding infill masonry walls to the existing frames of building block B_2. The strategic locations for application of retrofitted methods are shown in plan in Fig. 3.2 (b). The steel bracing are used in three patterns as shown in Fig. 4.1. These patterns are named as V Fig. 4.1 (a), Diamond Fig. 4.1(b) and Cross Fig. 4.1 (c). The infill masonry walls of two types are shown in Fig. 4.2. The complete infill masonry walls are added in two types viz. Infill masonry wall type1, Fig. 4.2 (a) and Infill masonry wall type 2, Fig. 4.2 (b). The type 1 Infill walls are added to edge frame in middle two bays as well as ground floor in x-direction (but, only middle bay in z-direction). In type 2 the infill walls are added to the end bays of edge frame along with ground floor bays. In x-direction the edge frames are selected for retrofitting as described earlier. All the three patterns of steel bracing are used in x-direction but in z-direction only cross base pattern is used Fig 4.3. This is, because in z-direction the first bay has corridor opening etc. and, second and third bay are not suitable as explained in following section. Both types of infill walls are used in x-direction and only type 1 is used in z-direction.

4.1.1 RETROFITTING USING STEEL BRACING

The use of steel bracing is more common for resisting lateral forces due to wind and earthquake, in steel structures, industrial buildings and transmission towers. Many reinforced concrete building have been designed and retrofitted using steel braces in various patterns. Since the steel bracing add strength and stiffness to the inadequate building without much addition of mass. The rolled steel sections in different shapes and sizes are easily available in market. Further the less obstruction is created for light and air as compared to infill walls. Therefore, considering all these factors the steel bracing in three patterns (V, Diamond and Cross) are used in the study as described below:

(i) V-PATTERN STEEL BRACING [Fig.4.1 (a)]

This name is given to the pattern because the shapes created by two diagonal braces of adjacent bays, in a story on meeting, give resemblance with letter 'V' of English alphabet. The sections adopted on the basis of design are 2ISA150×115×12 @ 47.6 Kg/m connected back to back at a spacing of 12 mm, are used. The idea behind the use of this pattern is that during reversal in the direction of the forces, the braces in one bay will be in tension and other will be in compression and give additional stiffness in frame to resist lateral forces. This pattern is used in x-direction only. Three-dimensional view along with cross pattern brace in z-direction is given in Fig. 4.4.

(ii) DIAMOND PATTERN STEEL BRACE [Fig.4.1(b)]

This name is given to the pattern because the diamond shape is created when the diagonal braces of two bays in the adjacent stories placed. The rolled steel sections (i.e. ISA150×115×12 @ 47.6 Kg/m) connected back to back at an spacing of 12 mm in the similar way as for V-pattern. In this pattern the lateral force (tension or compression) is expected to be resisted by equal number of diagonal braces in both the bays in case of reversal, not like V-pattern where all diagonal member of one bay will be in tension or compression. This pattern is also used in x-direction frames only. Three-dimensional view of building block using this pattern is shown in Fig. 4.5.

(iii) CROSS PATTERN STEEL BRACE [Fig. 4.1 (c)]

The name of this pattern has obvious meaning. The two diagonal steel braces in a bay of a particular story intersect each other. The section used for this pattern on the basis of design is 2ISA100×100×10 @ 29.8 Kg/m connected back to back at spacing of 10 mm. The idea behind the use of this pattern is that by decreasing the length of diagonal brace the size of the sections can be reduced. Since the diagonal braces are available in both the directions therefore this pattern can be used in both x and z-directions. The three dimensional view of building block using this pattern is shown in Fig. 4.6.

4.1.1.1 CONNECTIONS USED FOR STEEL BRACING

The connection of steel sections with reinforced concrete frames is not an easy task. The connection used in the steel brace patterns are checked for incoming forces. The 10-mm thick steel plates are embedded at the end of beams on both faces (top and bottom). The holes were drilled in this plate to accommodate the threaded rod bolts, before placing in position. The T sections (ISST 250 @ 37.5 Kg/m) are attached to these plates by welding. The threaded rod bolts are inserted into the drilled holes and then bolted as per design. The diagonal steel brace ends grip the T section from both sides. The riveting is used to connect the brace with T section, which is attached to the end plates. The centroidal axis of braces, is expected to pass through the beam column joints. Keeping one diagonal brace continuous and cutting the other in two pieces makes the connections at intersection. These pieces of the diagonal are attached together by means of a flat steel plate. The ends of braces are riveted with connection plate of sufficient cross sectional area. The checks, for out of plane buckling and designs of diagonal steel braces and connections for cross pattern, are given in appendix A.1, with sample calculations. The connections for V and Diamond pattern adopted section are also checked, considering single diagonal brace as compression and tension member.

4.1.2 RETROFITTING USING INFILL MASONRY WALLS

A number of Reinforced concrete framed buildings have been retrofitted by adding infill masonry walls of concrete blocks. The infill walls are commonly added to the low-rise building upto 5 or 6 stories. Since the infill wall increase the mass of building and thus change in dynamic characteristics takes place. Further foundations may need strengthening due to increase in mass. The infill masonry walls constructed with bricks are used in this study. The strength of brick masonry (f_m=24Mpa) in terms of modulus of elasticity (E_m=600.f_m) is used in the study. The removal of 300 MM thick and 1100MM high spandrel wall is expected to compensate the additional increase in mass. The infill walls using concrete block masonry are used in this study. The small discrepancy in mass is not considered to give any major changes in results. The sample calculation for design of an infill masonry panel of ground floor in x-direction frame is given in appendix. The type of infill used for seismic retrofitting are described below:

(i) INFILL MASONRY WALL TYPE 1 [Fig. 4.2 (a)]

In this type the middle two bays are in x-direction are completely (i.e. without opening) infilled with brick masonry walls in between clear opening of frames after removing clear covers from beam and columns. The thickness of masonry wall in x-direction is taken as 200 MM and in z-direction as 250 MM. The elastic modulus E_m is taken as 14400 MPa. For construction, 1:2 (cement: sand) mortar can be used which is expected to give proper bond between masonry units. Later on the faces of infill walls will be plastered. Three-dimensional view of infill masonry wall type 1 for block B_2 is shown in Fig. 4.7 along with the infill in middle bay in z-direction.

(ii) INFILL MASONRY WALL TYPE 2 [Fig. 4.2(b)]

In this type the infill walls are added to in the end bays and ground floor in x-direction. The masonry is same as for type 1. The two types are considered to study the effects of change in location of infill walls. This type 2 is only provided in x-direction frames because the first bay in z-direction has corridor openings as mentioned earlier. The Fig. 4.8 shows the three dimensional view of infill masonry wall type 2.

4.2 ASSUMPTIONS IN RETROFITTING

The following assumptions are made in retrofitting:

(a) The infill masonry walls are modeled as 4-noded elements having thickness 200 MM in x-direction and 250 MM in z-direction and connected to centers of the respective joints while in actual they will be provided in clear openings.

(b) The V, diamond and cross pattern steel braces are connected their ends with T sections and connecting plates which are attached to the beam ends hence may cause some additional axial forces.

(c) The cross braces are designed for full length in tension and half-length in compression with an effective length factor 0.85.

(d) The retrofitting methods are supposed to maintain symmetry and avoiding torsion effects but the V-pattern may give torsion in the building.

(e) The steel braces are assigned as truss members assuming that the shear force and B.M. will not develop.

4.3 METHODOLOGY ADOPTED FOR SEISMIC RETROFITTING

The following methodology is adopted in seismic retrofitting of the object building:

1. First of all the capacity demand (C/D) ratios for response quantities are determined on the basis of existing and revised draft code provisions.
2. The members showing C/D ratios less than unities represent distress are considered as unsafe.
3. Since most of the members show C/D ratio less than 1, therefore it is considered to retrofitted entire frames instead of strengthening the individual member.
4. The spandrel walls were removed completely before adding steel braces or infill walls.
5. The member showing C/D ratio less than unity even after addition of steel bracing or infill masonry walls are suggested to strengthen individually.

4.4 METHOD OF ANALYSIS USED FOR RETROFITTING

The response spectrum analysis as described in previous chapter, using STAAD-III (Research Engineers Inc., 1997) software package is also used in dynamic analysis of retrofitted frames. The building is analyzed as 2D frames in x and z directions. The steel braces were added to the existing frame and assigned as truss member in analysis. In the cross brace pattern of steel bracing, additional joints are created at intersection point of diagonal braces. The infill panels are added in the form of 4-noded elements. The mass of building is expected to remain unchanged before and after retrofitting, because the removal of 300-mm thick spandrel walls are assumed to compensate, the increase in mass due to steel braces and masonry infill walls.

The ground motion inputs in the form of defined response spectra of code were considered in longitudinal direction of frames. The responses are then combined as per SRSS rule.

4.5 DISCUSSIONS OF ANALYSIS RESULTS

The frames shown in Fig. 3.3 and 3.4 were analyzed after addition of steel bracing in three patterns (i.e. V, Diamond and Cross) or infill masonry wall of type 1 and type 2. The method of analysis is given in previous section 4.4. The results obtained from dynamic analysis of retrofitted frames are discussed and also compared with existing frame in x and z-direction as given in following sections.

4.5.1 RESULTS OF V-PATTERN STEEL BRACING

The results of elastic, linear dynamic analysis for retrofitted frames in x-direction using V-pattern steel bracing are discussed below:

(a) Time Periods

The time period corresponding to first 10 modes, for existing and retrofitted edge frames in x-direction are given Table 4.1(a). The fundamental time period for retrofitted frames reduces by 40% due to addition of V-pattern brace. The decrease in time periods represents stiffening of frame. The fundamental time period for existing building was 1.802 sec.

(b) Bending Moments

The capacity demand ratios for bending moments in beams of existing and retrofitted frames in x-direction are given in Table 4.17. The C/D ratio after using V-pattern steel bracing increases in all the members but the few members indicate distress (C/D<1). These unsafe C/D ratios in members are given in bold figures. Mid span sections of beams are safe as the C/D ratio for B.Ms. are greater than 1, for retrofitted frames as given in Table 4.19.

The bending moment capacity and demand in columns before and after retrofitting are given in Table.4.3. The B.M. in ground floor column of existing building which has C/D ratio 0.939 that is brought to 1.33 after retrofitting.

(c) Shear Forces

The base shears for existing and retrofitted frames in x-direction are given in Table 4.1(b). The base shears after addition of V-pattern steel brace have increased by 51.88% as compared to existing frames. This shows that the retrofitted frames attract more base shear as compared with existing frames which is due to stiffening of frames. The end shear forces in beams at section I, for existing and retrofitted edge frame in x-direction, are shown in Table 4.16. The table shows that the end shear forces has been reduced after retrofitting in all the beams. The variation of end shear forces in columns is given in Table 4.3.2. The shear forces in ground floor columns are given in Table 4.7. The capacity demand ratio determined for shear forces in ground floor column before retrofitting was 0.778 as given in Table 3.6 and becomes 2.089 after retrofitting as in Table 4.7, because the spandrel walls have been removed completely.

(d) Joint Displacements

The horizontal joint displacements at floor level joints for existing and retrofitted frames are given in Table 4.10. The joint displacement reduces by 25.6% at first floor level and 40.17% at roof level after retrofitting. These joint displacements are plotted in Fig.4.9. The maximum joint displacement in existing frame was at roof level and equals to 7.1285cm.

(e) Story Drifts

The story drifts for frames in x-direction are given in Table 4.12 and are also plotted in Fig. 4.12. The story drifts in lower stories are larger than that in top stories. The maximum drift in retrofitted frame is at third floor level, which has reduced by 44.49% as compared to the existing frame. The maximum drift is of the order of 0.001359 at third floor level.

(f) Axial Forces

The capacity and demands for axial forces in beams in x-direction are given in Table 4.4. The beams in middle bays show larger increase in axial forces than the beams of end bays. The axial load carrying capacity for beams are determined on the basis of minimum mid span reinforcement (i.e.2 bars of 20 mm diameter) and treating beams as column. Thus the beams were found safe to carry increased axial forces.

The axial force variation along height of frames for existing and retrofitted frames are given in Table 4.3.1 which shows 9% decrease in axial force for ground floor column after addition of V-pattern steel braces. The C/D ratio of axial forces in columns of ground floor are determined on the basis of capacity and demands as given in Table 4.5. The capacity demands ratio for axial force in ground floor columns after retrofitting is found greater than one, in spite of increase in axial forces in G/F columns.

4.5.2 RESULTS OF DIAMOND PATTERN STEEL BRACING

The results obtained from dynamic analysis of retrofitted frames using diamond pattern brace are given as follows:

(a) Time periods

The fundamental time periods for existing and retrofitted edge frames in x-direction are given in Table 4.1(a) due to addition of diamond pattern steel bracing. The table shows that fundamental time periods reduce from 1.802 sec to 1.086sec. It can be noticed that fundamental time period for this pattern and v-pattern brace has not much difference. Therefore the stiffness provided by both patterns to the frames in x-direction is approximately of the same order. However diamond pattern gives better stiffness than V-pattern.

(b) Bending Moments

The B.M. capacity demand ratio for beams in existing and retrofitted frames in x-direction are given in Table 4.17. The C/D ratios after using diamond patterns have increased in all members except few sections at ground and first floors. The C/D ratio less than one are shown in bold figures. The B.M. capacity and demands in columns at the end section are shown in Table 4.3. It can be seen that B.M. capacity of the end column at ground floor in existing frame was 6.04% less than the demands, and after retrofitting capacity demand ratio becomes 1.185. The variation of B.M. in the Table 4.3 shows that in the columns the demand reduces along height of building.

(c) Shear forces

The Table 4.1(b) shows the base shear for existing and retrofitted frames at edge in x-direction. The base shear has increased by 53.19% due to addition of the diamond pattern steel bracing. The shear forces in columns along height of building are given in Table 4.3.2. and the ground floor columns show that 11.01% reduction in demands as compared with existing frame. The end shear forces in beams at section I, for existing and retrofitted edge frame in x-direction, are shown in Table 4.16. The table shows that the end shear forces has been reduced after retrofitting in all the beams. The factored shear capacities and demands for ground floor columns of edge frame in x-direction are given in Table 4.7, which shows that all the ground floor columns are safe after retrofitting.

(d) Joint Displacements

The joint displacements for edge frame in x-direction are shown in Table 4.10 and are also plotted in Fig 4.9. The joint displacement at roof level is 4.084cm and it has reduced by 42.67% as compared to existing frame.

(e) Story Drifts

The story drifts for existing and retrofitted frames are given in Table 4.12. The values in this table represent 42.96% reduction in story drifts at roof level. The maximum story drift in retrofitted frame is at the third floor level i.e. 0.4080 cm. Therefore the maximum story drift is within permissible limit (1.24 cm). The Fig 4.12 shows the plot of the story drifts for existing and retrofitted edge frame in x-direction.

(f) Axial Forces

The capacity and demand for axial forces in beams of edge frame in x-direction are shown in Table 4.4, which represents that the beams have sufficient capacity to resist the demands. The variation of axial forces in columns of edge frames are shown in Table 4.3.1 The axial force demands in columns above seventh floor level for existing and retrofitted frames are nearly equal. The axial force capacity and demands in columns of existing and retrofitted frame at ground floor are given in Table 4.5. It can be observed from Table 4.5 that the axial force increased after the use of V-pattern brace but are less than capacities.

4.5.3 RESULTS OF CROSS PATTERN STEEL BRACING

The results of analysis of retrofitted frames using cross pattern steel bracing are given below with brief disscussions:

(a) Time Periods

The time periods corresponding to first ten modes as given in Table 4.1(a) for edge frame in x-direction represents that fundamental time period reduces by 50.11% after retrofitting the frame. In z-direction for frames at edge and middle the time periods are given in Table 4.2(a). The decrease in fundamental time periods for edge and middle frames decreases by 36.90% and 36.89% respectively.

(b) Bending moments

The bending moment capacity demand ratios for beams of edge frame in x-direction are given in Table 4.17. The values in the table shown in bold figures represent unsafe C/D ratios. C/D ratios after retrofitting using cross pattern steel bracing have increased in all beams except in one beam at ground floor and two beams at eleventh and twelfth floors. The C/D ratios of B.Ms. in beams of edge frame in z-direction are given in Table 4.18. The values show a decrease in C/D ratio in beams at eleventh and twelfth floor. The sections at mid spans in beams are found safe as given in Table 4.19.

The Bending moment capacity and demands in columns of edge frames are given in Table 4.3. It can be shown in this table that the B.M. demands using cross pattern are lesser as compared to other steel brace pattern (i.e. V and Diamond). The C/D ratio for ground floor column increased from 0.939 to 1.448. The B.Ms. in columns in Z-direction for existing and retrofitted frame are given in Table 4.8. The B.M. C/D ratios in G/F and F/F columns were less than one but after retrofitting are found greater than unity.

(c) Shear forces

The Table 4.1(b) gives the base shears for existing and retrofitted edge frames in x-direction. The base shear in retrofitted frame has increased by 75.51% as compared to existing frame. The base shears in z-direction frames at edge and middle after retrofitting have increased by 44.30% and 47.95% as given in Table 4.2(b). The end shear forces in beams at section I, for existing and retrofitted edge frame in x-direction, are shown in Table 4.16. The table shows that the end shear forces has been reduced after retrofitting in all the beams. Shear force capacity and demands in columns of edge frame are given in Table 4.7. The shear force capacities are greater than demands for columns of retrofitted edge frame in x-direction.

(d) Joint displacement

The joint displacements at floor level for edge frame in x-direction are given in Table 4.10 and the same are also plotted in Fig. 4.9. The maximum joint displacement in retrofitted frame is found at roof level is 3.3799cm. It can be noted from the Table 4.10 that the joint displacements at roof level and F/F level reduced by 51.59% and 41.39% respectively. The joint displacements in z-direction edge and middle frames are given in Table 4.9. These joint displacements are also plotted in Fig. 4.10 and 4.11 for edge and middle frames in z-direction.

(e) Story drift

The story drifts for edge frame in x-direction are given in Table 4.12. This table shows that the maximum drift in x-direction frame is 0.3034 cm, which is within permissible limit (i.e. 1.24cm). The Fig. 4.12 show the story drifts for existing and retrofitted edge frame in x-direction. The story drifts in z-direction for edge and middle frames are given in Table 4.13 and the same are plotted in Fig.4.13 and 4.14 respectively. The story drifts at roof level have increased in edge frames by 21.06 and 21.99% in middle frame.

(f) Axial forces

The axial forces in beams of edge frames in x-direction are given in Table 4.4 for existing and retrofitted frames. The axial force capacities in retrofitted frames are greater than unity for all beams. Table 4.9 shows the axial forces in beams of edge frames in z-direction.

The axial force variation in columns for edge frames in x-direction are given in Table 4.3.1. It can be seen that the demands due to cross pattern bracing are lesser as compared to other patterns. The Table 4.5 shows that the columns at G/F in edge frame are safe.

4.5.4 RESULTS OF INFILL MASONRY WALL TYPE 1

The results obtained from dynamic analysis of retrofitted frames using infill masonry wall type 1 are given below:

(a) Time periods

The time periods for existing and retrofitted edge frame in x-direction are given in Table 4.1(a). The fundamental time periods in x-direction has decreased by 72.80% as compared with existing frame. Table 4.2(a) gives the time periods for edge and middle frames in z-direction. The time periods are reduced by 55.27% and 55.25% in retrofitted edge and middle frames in z-directions respectively.

(b) Bending Moments

The bending moment capacity demand ratios for existing and retrofitted frames in x-direction are given in Table 4.17. The capacity demand ratios have increased in majority of beams except few sections. The bold figures in table represent the unsafe C/D ratios. The C/D ratios for B.Ms. in beams of z-direction are given in Table 4.18. The B.Ms. capacity demand ratios at mid span of beams in x-direction edge frame are greater than 1 as shown in Table 4.19.

The variations of B.Ms. in columns of edge frame in x-direction are shown in Table 4.3. It can be seen that the demands are less than capacities at each floor level. The G/F columns are safe as given in Table 4.6. The variation of B.Ms. in columns of edge frame in z-direction is given in Table 4.8. Though the B.M. in columns increases towards top but the capacities are larger than demands at each floor level.

(c) Shear Forces

The base shears for existing and retrofitted frames in x-direction are given in Table 4.1(b). The base shear increased by 176% as compared with existing frame. The Table 4.2(b) shows the base shears in existed and retrofitted frames in z-direction. The base shears after retrofitting have increased by 76.81% and 85.10% for edge and middle frames respectively. The end shear forces in beams at section I, for existing and retrofitted edge frame in x-direction, are shown in Table 4.16. The table shows that the end shear forces has been reduced after retrofitting in all the beams. The Table 4.7 gives shear forces in G/F columns, the demands in retrofitted frames are less than capacities. The variations of shear forces in columns of frame in x-direction are given in Table 4.3.2. It can be noted that demands in ground floors column have reduced by 61.28%.

(d) Joint displacements

The joint displacements for edge frame in x-direction are given Table 4.10 and are also plotted in Fig. 4.9. The joint displacements at roof level reduced by 68.10%. The Table 4.11 gives the joint displacements in z-direction for edge and middle frames. The joint displacement at roof level have reduced by 54.07% and 53.99% for edge and middle frames in z-direction respectively. The Fig. 4.10 and 4.11 show plot for joint displacements given in Table 4.11.

Story Drifts

The Table 4.12 shows story drifts for edge frame in x-direction. The plots of these story drifts are given in Fig. 4.12. The maximum story drift equal to 0.1923cm is at ninth floor level. The story drifts in z-direction frames are given in Table 4.13 and are also plotted in Fig.4.13 and 4.14 for edge and middle frame in z-direction respectively. The maximum story drifts are 0.2818cm at tenth floor and 0.3664cm at eleventh floor in retrofitted frame at edge and in middle. All the story drifts are found within permissible limit. (i.e. 0.004 times story height 1.24 cm).

(f) Axial Forces

The axial forces in beams of edge frame in x-direction are given in Table 4.4. The table shows that the axial forces in beams of middle bay are more than that in end bay but are safe. The variation of axial forces in columns of edge frame in x-direction are given in Table 4.3.1., Which shows that the demands in retrofitted frame are lesser than capacities. The ground floor columns are safe for axial forces. As per table 4.5 it can be seen that the axial force in end columns at G/F decreases by 16.45% as compared with demands in existing frame.

4.5.4 RESULTS OF INFILL MASONRY WALL TYPE 2

The following results are obtained from dynamic analysis of retrofitted frames using complete infill of type 2.

(a) Time Period

The time periods corresponding to first 10 modes for existing and retrofitted frame are given in Table 4.1(a). The fundamental time period has been decreased by 65.31% after using in fill type 2.

(b) Bending Moments

The bending moment capacity demand ratio for existing and retrofitted frames in x-direction is given in Table 4.17. The majority of members have C/D ratios greater than 1 except few sections shown in bold figures. The column bending moments are given in Table 4.3, the demands in this table are less than capacities.

(c) Shear Forces

The Table 4.1(b) shows base shear for existing and retrofitted frames. The base shears has increased by 100.2% as compared to existing frame. The end shear forces in beams at section I, for existing and retrofitted edge frame in x-direction, are shown in Table 4.16. The table shows that the end shear forces has been reduced after retrofitting in all the beams. The factored shear forces for ground floor columns are given in Table 4.7. It can be seen that shear force demands at ground floor end columns is minimum among all other methods used in this study.

(d) Joint Displacements

The joint displacements for existing and retrofitted frames in x-direction are given in Table 4.10 and are also plotted in Fig4.9. The maximum joint displacement occurs at the roof level, which is equal to 2.749 cm. The joint displacement has reduced by 68.10% as compared with existing frame.

(e) Story Drifts

The story drifts for existing and retrofitted edge frames in x-direction are given in Table 4.12 and are plotted in Fig.4.12. The maximum drift in retrofitted frame is found at eleventh floor, which is equal to 0.2527cm and is within permissible limit. It can be observed from the Table 4.12 that the story drifts at roof level has increased by 88.7% but is within permissible limit.

(f) Axial Forces

The axial forces in beams of existing and retrofitted edge frames in x-direction are given in Table 4.4. The axial force demands in G/F beams are greater than that for complete infill type 1. The Table 4.3.1 shows the axial forces in columns along height of building. It can be observed from the Table 4.3.1, that the B.Ms. demand in columns of retrofitted frame increases upto eighth floor with respect to the lower floor level. The ground floor columns are safe for axial force demands. Table 4.5 shows that the axial force after using Infill masonry wall type 2 for retrofitting the axial force in end columns increases by 29.39%

while it decreases in middle columns. Eventually the axial force C/D ratios in columns are found safe for existing reinforcement.

4.6 RELATIVE MERITS AND DEMERITS OF RETROFITTING METHODS USED IN THIS STUDY

Two methods of retrofitting used in the present study are compared on the basis of their performance and approximate quantities of materials required for construction. For the comparison of methods only edge frame in x-direction is considered because all three studied patterns of steel bracing or infill masonry wall of two types are provided in x-direction only. The Table 4.20 gives the comparison of the performance and material required in different methods. The materials required as given in Table 4.20 is for single edge frame in x-direction.

(a) Retrofitting using steel bracing

Relative merits and demerits of steel bracing in three patterns (V, Diamond and Cross) are discussed on the basis of results of analysis and Table 4.20, and given below:

Among the three pattern of steel bracing cross pattern shows better stiffening, reduction in story drift at ground floor and joint displacements at roof level. The number of sections of beams, which require individual strengthening are less than as in V and diamond patterns. However, as per Table 4.20 the quantity of steel in diagonal brace and other material required for connections is more than that for V and diamond. Though the section of steel used in diagonal brace in cross pattern is lighter than that in V and diamond yet the saving in steel couldn't achieved as they are two in number. Diamond pattern is better than V-pattern because it gives better reduction in fundamental time period due to increase in the stiffness of the existing frame. The reduction in story drifts at ground floor and joint displacement at roof level is comparatively better than V-pattern. Further more the number of beam sections requiring individual retrofitting is less than that in V-pattern. The Table 4.20 indicates that in spite of same quantity of steel used in diagonal braces, the diamond pattern shows considerable saving in material used for connections (i.e. less number of connection plates, and threaded rod bolts etc.). V-pattern gives poor performance as more number of beam sections require individual strengthening, in addition to this, quantity of material required is more than that in diamond pattern.

On the basis of above discussion it is clear that the cross pattern shows better performance but is expensive as compared with V and diamond pattern. However diamond pattern gives moderate performance and economy as compared to V and cross pattern. The V pattern shows relatively poor performance among the three patterns.

(b) Retrofitting using infill masonry walls

On the basis of result of analysis and comparisons given in Table 4.20, following merits and demerits can be pointed out:

The infill masonry wall of type 1 gives better reduction in fundamental time period meaning thereby the better addition to stiffness of the existing frame as compared with type 2. The reduction (as compared with the existing frame) in joint displacement at roof level is more in type 1 than type 2. Story drifts at ground level for type 1 is more than type 2 but above third floor type one shows better reduction in story drifts as given in Table 4.12. However the story drifts are within permissible limits. The numbers of beam sections require individual strengthening is less for infill masonry wall type 1 than type2.

Considering these results the infill masonry wall of type 1 is considered better than type 2. The following points can also be considered to compare the two methods:

- The obstruction created to light, cross ventilation and view is more in case of infill masonry walls (i.e. walls without opening) than the steel bracing.
- The availability of materials and equipment needed for the construction of steel bracing or infill masonry walls. It is clear that the steel sections are easily available in market in various sizes. However, the masonry of desired strength can be adopted on the basis of stress level in the member.

CHAPTER-5

SUMMARY AND CONCLUSIONS

5.0 GENERAL

In the present dissertation, seismic retrofitting of 14-storied reinforced concrete building has been studied. . The results obtained in the study are summarized here in this chapter. The important buildings in severe zones of Indian subcontinent need seismic evaluation to study their seismic performance. The buildings found inadequate to future probable earthquake should be retrofitted by adopting efficient retrofitting strategies to mitigate anticipated hazards to life and property of occupants.

5.1 SUMMARY

The existing reinforced concrete building in seismic zone IV, designed as per IS:1893-1984 using limit state method and following design code IS:456-1978 shows inadequate response according to revised draft provisions. The seismic evaluation of building was carried out by 2D elastic linear dynamic analysis, using computer software package (STAAD-III, 1997). Majorities of beams in the existing edge frame as well as middle frame are found weak for B.Ms. at support and mid span. Short column failure is also detected in the columns in edge frames, upto fifth floor level, due to presence of spandrel walls. The maximum values of story drifts in existing as well as retrofitted frames were found within permissible limit.

Two methods were adopted for seismic retrofitting such as steel bracing in three patterns and infill masonry panels in two types. The reanalysis of retrofitted frames show that the C/D ratio has increased due to addition of steel bracing or infill masonry walls in majority of members. Few members still need individual strengthening to make C/D ratios greater than one. The relative performance of the methods of retrofitting has also been studied and conclusions drawn on the basis of study are given in following section 5.2.

5.2 CONCLUSIONS

The following conclusions may be drawn on the basis of present study and reviewed literature:

1. The existing building in seismic zone IV, designed and constructed using IS: 456-1978 and analysed as per existing seismic code IS:1893-1984 is found inadequate for revised draft code provisions.
2. The seismic performance of the building having ordinary moment resisting frames can be improved by adding steel bracing or infill masonry walls.
3. Cross-pattern steel bracing, shows better performance than V and diamond pattern, but is relatively expensive. However, diamond pattern gives moderate performance as compared with V and cross pattern.
4. The infill masonry wall of type1 gives better performance than infill masonry wall type 2 for the same amount of infill material used.
5. Addition of infill masonry wall is better than steel bracing method of retrofitting but creates more obstruction to light, cross ventilation and view, as compared to steel bracing method.
6. The spandrel walls may create short column failure therefore they should be removed or separated from structural system to achieve proper frame action.

5.3 SCOPE FOR FURTHER STUDY

The problems related to seismic retrofitting of multistoried reinforced concrete buildings can be further studied for following additional considerations:

(a) The foundation effects and soil-structure interaction.

(b) The seismic evaluation and reanalysis of retrofitting methods used in the present study i.e. addition of steel bracing or infill masonry panels at strategic locations, can be further studied for 3D model.

(c) Using infill masonry walls at few floor levels of selected bays instead of all floors.

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TABLES

Table 2.1 Brief comparison of existing code IS:1893-1984 and revised draft code

S.No.	Particulars	IS:1893-1984	Revised draft
1.	Seismic zones	Five zones I, II, III, IV, and V	Four zones II, III, IV and V
2.	necessity of dynamic analysis method	a) For buildings of height greater than 40 m, in zones III, IV, and V, b) Greater than 90 m in zones II and I. c) Desirable for irregular buildings using RSA.	a) Regular building greater than 40 m in zones IV and V, and b) Greater than 90 m in height in zones II and III. c) For irregular buildings lesser than 40 m in zone II and III.
3.	Seismic base shear (V_B)	$KC\alpha_h W$	$A_h W$
4.	Horizontal seismic coefficient	$\alpha_h=\beta I\alpha_o$ (seismic coefficient method)	$A_h = \dfrac{\left(\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)}{\left(\frac{R}{I}\right)}$
		$\alpha_h=\beta IF_oS_a/g$ (Response spectrum method)	
5.	Vertical seismic coefficient	Half the horizontal seismic coefficient.	Two third of the horizontal seismic coefficient.
6.	Importance factor	For important buildings = 1.5	Same as in Existing code.
		For other buildings = 1.0	
7.	Soil foundation factor (β)	Depends on soil foundation system.	Included in two separate response spectra for rock site and soil site.
8.	Seismic zone factor	F_o, For zones I to V are 0.05, 0.1, 0.2, 0.25 and 0.4 respectively.	Z, is refers to zero period acceleration value for MCE, table () MCE/DBE = 2
9.	Performance factor	K, depends upon type of construction, brittleness and ductility The values range from 1 to 1.6 for most ductile MRF to brittle masonry walls.	R response reduction factor with values ranging from 2 to 5. For O.M.R.F. =3 For S.M.R.F. =5
10.	Fundamental time period, T (Second)	T=0.1n (for MRF without bracing or shear walls).	$T=0.075\,(H)^{0.75}$ (for MRF without bracing and shear walls. $T=0.085\,(H)^{0.75}$ (for steel frames)
		$T = \dfrac{.09H}{\sqrt{d}}$ (for other buildings) H= the total height of building in m. d= maximum plan dimension at base in the direction parallel to the applied seismic force in m.	Same as in existing code. (for other buildings)
11.	Storey drift	Shall not exceed .004 times the storey height.	Same as in existing code.
12.	Distribution of base shear along building height at floor levels	$Q_i = V_B \dfrac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$ (parabolic distribution)	Same as in existing code.
13.	Torsional response	Consideration to be made for the increase in shear due to an eccentricity between CM and CR.	Provision for accidental torsion are made.
14.	Building configuration	Few irregular buildings like plaza type, flexible first storey and building founded on hilly slopes.	Plan and vertical irregularities are well defined.

Table 3.0 Description of building

S. No.	Particulars	Details/Values
1	TYPE OF BUILDING	HOSPITAL
2	NUMBER OF STOREYS	14 (G+13)
3	SPACING OF FRAMES IN X-DIRECTION	6.5 M
4	SPACING OF FRAMES IN Z-DIRECTION	6.0 M
5	NUMBER OF BAYS IN X-DIRECTION	4
6	NUMBER OF BAYS IN Z-DIRECTION	3
7	JOINTS RESTRAINED IN X-DIRECTION FRAMES	5
8	JOINTS RESTRAINED IN Z-DIRECTION FRAMES	4
9	EACH STOREY HEIGHT	3.1 M
10	SIZE OF ALL BEAMS	500×500 MM
11	SIZE OF COLUMNS	700×700 MM
12	LOADING CLASS	250
13	% OF LIVE LOAD CONSIDERED	25
14	GRADE OF CONCRETE USED IN	
15	ALL BEAMS	M15
16	COLUMNS UPTO 5^{TH} FLOOR	M 25
17	COLUMNS ABOVE	M 20
18	GRADE OF STEEL	Fe-415

Table 3.1 Dynamic characteristics of existing building frames

Mode No.	X-Direction		Z-Direction	
	Time Period (Seconds)	Mass Participation (%)	Time Period (Seconds)	Mass Participation (%)
1	1.80187	78.87	1.74522	78.65
2	0.57763	9.84	0.55836	10.06
3	0.32280	3.85	0.31066	3.86
4	0.21262	2.19	0.20464	2.20
5	0.15137	1.45	0.14570	1.45
6	0.11336	1.04	0.10915	1.04
7	0.08813	0.78	0.08488	0.77
8	0.07070	0.59	0.06811	0.59
9	0.05873	0.46	0.05620	0.46
10	0.04929	0.36	0.04752	0.36

Table 3.2 Bending moment capacity demand ratio for beams of edge frame in x-direction

Beam No.	Capacitiy C (KN×M)			Demand D (KN×M)			C/D Ratio		
	Node I	Middle	Node J	Node I	Middle	Node J	Node I	Middle	Node J
2001	205.09	172.07	205.09	280.54	223.23	273.99	0.731	0.771	0.749
2002	205.09	139.75	205.09	275.76	218.15	274.98	0.744	0.641	0.709
2011	242.57	172.07	242.57	338.63	274.84	320.25	0.716	0.626	0.757
2012	242.57	172.07	242.57	327.98	270.73	327.25	0.740	0.636	0.741
2021	242.57	172.07	242.57	353.51	284.82	325.31	0.686	0.604	0.746
2022	242.57	172.07	242.57	340.15	282.57	338.72	0.713	0.609	0.716
2061	242.57	172.07	242.57	323.08	240.54	267.22	0.751	0.715	0.908
2062	242.57	172.07	242.57	303.54	245.44	300.36	0.799	0.701	0.808
2071	242.57	135.05	195.01	310.45	225.48	249.73	0.781	0.599	0.781
2072	195.01	135.05	195.01	289.81	231.59	286.27	0.673	0.583	0.681
2111	195.01	139.75	172.07	234.14	142.92	161.04	0.833	0.978	**1.068**
2112	172.07	94.15	172.07	210.57	151.65	205.61	0.817	0.621	0.837
2121	195.01	139.75	172.07	205.86	114.99	134.14	0.947	0.819	**1.283**
2122	172.07	94.15	172.07	182.60	124.87	179.01	0.942	0.754	0.961
2131	195.01	139.75	172.07	161.61	91.24	112.07	**1.210**	**1.032**	**1.535**
2132	172.07	94.15	172.07	146.25	96.69	141.26	**1.177**	0.974	**1.218**

- Bold figures represent safe section of member.

Table 3.3 Bending moment capacity demands ratio for beams of edge frames in x-direction as per site spectra and revised draft spectra

Beam No.	Capacity (KN×M)		Demand As Per Site Spectra (KN×M)		Demand As Per Revised Draft (KN× M)		C/D Ratio	
	Node I	Node J	Node I	Node J	Node I	Node J	Node I	Node J
2001	205.09	205.09	197.19	192.67	280.54	273.99	0.731	0.749
2002	205.09	205.09	195.11	194.38	275.76	274.98	0.744	0.709
2011	242.57	242.57	229.06	213.22	338.63	320.25	0.716	0.757
2012	242.57	242.57	221.15	220.44	327.98	327.25	0.740	0.741
2021	242.57	242.57	238.92	213.31	353.51	325.31	0.686	0.746
2022	242.57	242.57	227.40	225.99	340.15	338.72	0.713	0.716
2061	242.57	242.57	230.87	177.15	323.08	267.22	0.751	0.908
2062	242.57	242.57	209.42	206.34	303.54	300.36	0.799	0.808
2071	242.57	195.01	225.86	167.12	310.45	249.73	0.781	0.781
2072	195.01	195.01	199.20	202.64	289.81	286.27	0.673	0.681
2111	195.01	172.07	191.31	119.28	234.14	161.04	0.833	**1.068**
2112	172.07	172.07	163.41	158.56	210.57	205.61	0.817	0.837
2121	195.01	172.07	177.15	119.84	205.86	134.14	0.947	**1.283**
2122	172.07	172.07	148.97	145.49	182.60	179.01	0.942	0.961
2131	195.01	172.07	143.14	95.19	161.61	112.07	**1.210**	**1.535**
2132	172.07	172.07	124.78	125.69	146.25	141.26	**1.177**	**1.218**

- Bold figures represent safe section of member.

Table 3.4 Bending moment capacity demand ratio for beams of middle frames in x-direction

Beam No.	Capacity C (Kn×M)			Demand D (Kn×M)			C/D Ratio		
	NODE I	MIDDLE	NODE J	NODE I	MIDDLE	NODE J	NODE I	MIDDLE	NODE J
2001	326.48	242.57	326.48	412.94	315.70	403.38	0.791	0.768	0.809
2002	326.48	205.09	326.48	406.86	308.43	405.66	0.802	0.665	0.802
2011	384.39	242.57	384.39	490.68	382.55	461.31	0.783	0.634	0.833
2012	384.39	242.57	339.05	474.72	376.76	473.49	0.810	0.644	0.716
2021	384.39	242.57	384.39	511.92	395.45	465.82	0.751	0.614	0.825
2022	384.39	242.57	339.05	490.64	392.12	488.22	0.783	0.619	0.694
2061	384.39	242.57	293.70	479.31	338.98	385.54	0.802	0.716	0.762
2062	293.70	242.57	326.48	444.23	344.84	438.88	0.661	0.703	0.744
2071	384.39	242.57	293.70	462.72	318.18	360.56	0.831	0.762	0.815
2072	293.70	242.57	326.48	425.29	325.71	419.33	0.691	0.745	0.779
2111	293.70	195.01	205.09	367.84	212.27	243.82	0.798	0.919	0.841
2112	293.70	195.01	293.70	323.77	223.01	315.41	0.633	0.874	0.931
2121	293.70	195.01	205.09	332.00	176.91	210.04	0.885	**1.102**	0.976
2122	293.70	195.01	293.70	287.84	189.08	281.77	0.713	**1.103**	**1.042**
2131	293.70	195.01	205.09	261.94	143.49	178.27	**1.121**	**1.359**	**1.150**
2132	293.70	195.01	293.70	232.47	149.07	233.77	0.882	**1.308**	**1.256**

- Bold figures represent safe section of member.

Table 3.5 Bending moment capacity demand ratio for beams of edge frame in z-direction

Beam No.	Capacitiy C (Kn×M)			Demand D (Kn×M)			C/D Ratio		
	Node I	Middle	Node J	Node I	Middle	Node J	Node I	Middle	Node J
2001	195.01	135.05	172.01	261.29	214.98	246.05	0.746	0.628	0.699
2002	172.01	135.05	172.01	252.86	208.79	252.86	0.680	0.647	0.680
2011	242.57	172.01	205.09	317.58	265.65	300.38	0.764	0.648	0.683
2012	205.01	172.01	205.09	304.54	262.68	307.44	0.673	0.655	0.667
2021	242.57	172.01	205.09	332.49	276.08	304.51	0.730	0.623	0.673
2022	205.01	172.01	205.09	318.35	274.68	318.76	0.644	0.626	0.643
2061	242.57	172.01	205.09	300.29	234.41	247.00	0.808	0.743	0.830
2062	205.01	172.01	205.09	281.38	238.41	281.83	0.729	0.721	0.727
2071	242.57	172.01	205.09	287.62	216.50	229.87	0.843	0.795	0.892
2072	205.01	172.01	205.09	267.71	224.84	268.14	0.766	0.765	0.765
2111	172.01	94.14	135.05	209.54	132.97	141.01	0.821	0.708	0.958
2112	172.01	94.14	135.05	186.81	144.08	187.20	0.921	0.653	0.721
2121	172.01	94.14	135.05	181.43	104.34	112.43	0.948	0.837	**1.201**
2122	172.01	94.14	135.05	159.09	116.63	159.48	**1.082**	0.807	0.847
2131	172.01	94.14	135.05	140.22	80.06	89.67	**1.227**	**1.176**	**1.506**
2132	172.01	94.14	135.05	125.71	89.21	126.08	**1.368**	**1.055**	**1.071**

- Bold figures represent safe section of member.

Table 3.6 Shear force capacity demand ratio for bottom section of columns of edge frame in x-direction

Column No.	Capacity	Demand as per analysis	Demand as per moment capacity of beam	Demand as per short column due to spandrel walls	C/D Ratio
	KN	KN	KN	KN	
1002	287.70	149.33	180.69	369.62	0.778
1012	287.70	157.97	219.09	327.50	0.878
1022	287.70	159.71	219.09	332.06	0.866
1032	287.70	154.97	219.09	323.32	0.890
1042	287.70	154.97	219.09	311.28	0.924
1052	287.70	148.67	219.09	326.98	0.879
1062	287.70	141.71	219.09	282.31	**1.019**
1072	287.70	134.38	180.69	266.20	**1.087**
1082	287.70	126.58	180.69	248.29	**1.158**
1092	287.70	117.98	176.14	227.53	**1.264**
1102	287.70	108.05	155.12	202.39	**1.422**
1112	287.70	96.04	155.12	171.11	**1.681**
1122	287.70	81.04	155.12	137.13	**1.854**
1132	287.70	64.76	155.12	88.55	**1.854**

- The columns upto fifth floor are unsafe from short column failure point of view.

Table 3.7 Shear force capacity demand ratio for beams of edge frame in z-direction

Beam No.	Capacitiy (Kn)		Demand (Kn)		C/D ratio	
	Node I	Node J	Node I	Node J	Node I	Node J
2001	265.58	265.58	156.54	155.77	**1.696**	**1.700**
2002	265.58	260.98	155.19	154.96	**1.710**	**1.710**
2011	269.98	269.98	174.17	171.10	**1.550**	**1.580**
2012	269.98	269.98	171.32	171.32	**1.580**	**1.580**
2021	269.98	269.98	178.73	171.65	**1.510**	**1.580**
2022	269.98	269.98	175.07	174.64	**1.540**	**1.550**
2061	269.98	269.98	169.51	153.64	**1.590**	**1.760**
2062	269.98	260.98	163.88	162.91	**1.650**	**1.650**
2071	269.98	260.98	165.67	148.20	**1.640**	**1.760**
2072	260.98	260.98	159.61	158.59	**1.640**	**1.650**
2111	269.98	256.48	142.46	121.63	**1.840**	**2.110**
2112	256.48	256.48	135.23	133.77	**1.900**	**1.920**
2121	269.98	256.48	133.84	112.16	**1.960**	**2.290**
2122	256.48	256.48	126.70	125.67	**2.020**	**2.040**
2131	269.98	256.48	115.19	100.81	**2.270**	**2.540**
2132	256.48	256.48	110.18	110.55	**2.330**	**2.320**

- Bold figures represent safe section of member.

Table 4.1(a) Time periods for existing and retrofitted edge frame in x-direction

Mode Number	Time Periods (Seconds)					
	Existing Frame	V-Pattern brace	Diamond Pattern brace	Cross Pattern brace	Infill Masonry Wall Type 1	Infill Masonry Wall Type 2
1	1.80187	1.08012	1.08610	0.89900	0.49000	0.62500
2	0.57763	0.36252	0.35380	0.28870	0.48843	0.67404
3	0.32280	0.20639	0.20144	0.16088	0.11960	0.14374
4	0.21262	0.14289	0.14007	0.11275	0.05885	0.06490
5	0.15137	0.10716	0.10556	0.08610	0.04184	0.04212
6	0.11336	0.08444	0.08359	0.06969	0.03380	0.03977
7	0.08813	0.06867	0.06815	0.05773	0.03055	0.03740
8	0.07070	0.05724	0.05764	0.04920	0.03034	0.03259
9	0.05873	0.04868	0.04897	0.04262	0.02961	0.03137
10	0.04929	0.04190	0.04225	0.03955	0.02952	0.02813

Tables 4.1(b) Base shears for existing building and retrofitted frame at edge in x-direction

Particulars of Frames	Base Shear
	Kn
Existing Frame	449.31
Retrofitted With V- Pattern Steel Brace	682.43
Retrofitted With Diamond Pattern Steel Brace	688.32
Retrofitted With Cross Steel Brace	788.60
Retrofitted With Infill Masonry Wall Type 1	1240.73
Retrofitted With Infill Masonry Wall Type 2	899.58

Table 4.2(a) Time periods for existing and retrofitted frames in z-direction

Mode Number	Time Periods (Seconds)					
	Edge Frame			Middle Frame		
	Existing	Cross Pattern	Infill wall Type 1	Existing	Cross Pattern	Infill wall Type 1
1	1.74522	1.10120	0.78060	2.21160	1.39570	0.98960
2	0.55836	0.34220	0.17672	0.70762	0.43356	0.21517
3	0.31066	0.18482	0.07939	0.39372	0.23419	0.09536
4	0.20464	0.12694	0.07939	0.25936	0.16082	0.06001
5	0.14570	0.09535	0.05034	0.18466	0.12018	0.04498
6	0.10915	0.07567	0.03779	0.13833	0.09584	0.03753
7	0.08488	0.06206	0.03139	0.10755	0.07858	0.03317
8	0.06811	0.05219	0.02761	0.08633	0.06607	0.03080
9	0.05620	0.04475	0.02535	0.07124	0.05665	0.03072
10	0.04752	0.03885	0.02511	0.06023	0.04920	0.03067

Table 4.2(b) Base shears for frames in z-direction

Particulars of Frames	Edge Frame	Middle Frame
	Base Shear (KN)	Base Shear (KN)
Existing Building	344.22	449.45
Retrofitted With Infill Wall	608.64	831.94
Retrofitted With Cross Brace	496.72	664.95

Table 4.3 Bending moments at bottom section of columns of edge frame in x-direction

Column No.	Capacity	Demand	Retrofitted With V-Pattern Brace	Retrofitted With Diamond Pattern Brace	Retrofitted With Cross Pattern Brace	Retrofitted With Infill Masonry Wall Type 1	Retrofitted With Infill Masonry WallType 2
	(Kn×M)	(Kn×M)	(Kn×M)	(Kn×M)	(Kn×M)	(Kn×M)	(Kn×M)
1003	421.37	448.46	315.95	355.56	290.85	75.17	37.54
1013	421.37	313.60	160.61	137.05	85.17	59.48	42.41
1023	421.37	259.11	123.60	115.85	81.85	57.25	60.79
1033	421.37	241.24	116.02	115.66	72.12	56.19	79.59
1043	421.37	238.27	111.23	100.03	68.84	55.07	89.24
1053	421.37	229.42	105.46	104.96	65.39	53.47	97.28
1063	414.40	218.53	99.25	88.71	61.59	51.34	103.27
1073	414.40	206.88	92.82	92.36	57.57	48.66	107.39
1083	414.40	194.63	86.16	76.21	53.31	45.40	109.83
1093	414.40	181.33	79.04	79.23	48.71	41.54	110.79
1103	414.40	166.01	71.11	61.29	43.55	37.01	110.54
1113	414.40	146.62	61.51	62.94	37.44	33.07	110.99
1123	414.40	118.99	50.82	42.55	32.13	27.71	107.52
1133	414.40	84.65	17.90	15.61	15.38	28.30	115.16

Table 4.3.1 Variation of axial forces in columns along height of edge frame in x-direction

Column No.	Existing Building	Retrofitted With V-Pattern Brace	Retrofitted With Diamond Pattern Brace	Retrofitted With Cross Brace	Retrofitted With Infill Masonry Wall Type 1	Retrofitted With Infill Masonry WallType 2
	KN	KN	KN	KN	KN	KN
1001	2181.28	2000.79	2000.56	1972.48	1774.32	2822.48
1011	2029.78	1861.63	1862.36	1838.15	1689.83	2690.63
1021	1862.49	1716.17	1717.41	1702.28	1557.61	2400.72
1031	1691.03	1568.27	1570.55	1564.56	1425.62	2130.80
1041	1519.62	1428.61	1425.55	1425.21	1296.71	1871.19
1051	1349.73	1287.45	1284.72	1272.04	1169.59	1616.78
1061	1181.90	1144.97	1142.53	1142.26	1040.49	1369.12
1071	1016.49	1001.33	999.23	998.99	909.93	1130.03
1081	854.57	856.69	854.89	854.71	778.40	901.00
1091	709.44	711.21	709.86	709.58	646.34	583.85
1101	563.63	565.04	586.86	563.76	514.11	481.78
1111	417.23	418.30	417.46	417.37	382.06	301.04
1121	270.25	270.99	270.41	270.40	250.32	154.91
1131	123.40	123.80	123.53	123.51	118.90	58.12

Table 4.3.2 variation of shear forces along height at bottom section in columns of edge frame in x-direction

Column No.	Existing Building	Retrofitted With V-Pattern Brace	Retrofitted With Diamond Pattern Brace	Retrofitted With Cross Brace	Retrofitted With Infill Masonry Wall Type 1	Retrofitted With Infill Masonry Wall Type 2
	KN	KN	KN	KN	KN	KN
1001	141.89	137.72	126.26	121.23	54.93	14.47
1011	134.24	94.05	99.07	76.78	45.56	16.15
1021	125.18	84.97	79.99	72.61	51.44	26.25
1031	120.06	85.94	87.35	76.66	55.33	27.98
1041	116.04	86.22	83.17	78.68	58.21	29.50
1051	112.44	86.17	87.42	80.06	60.51	30.79
1061	108.92	85.68	83.74	80.84	62.25	31.75
1071	105.26	84.72	85.63	81.03	63.47	32.42
1081	101.24	83.19	82.36	80.61	64.20	32.81
1091	96.55	81.00	81.50	79.54	64.51	32.95
1101	90.67	77.89	78.27	77.69	64.47	32.84
1111	83.06	73.71	73.80	74.96	63.56	32.44
1121	75.14	70.25	72.15	74.07	63.22	37.31
1131	76.99	77.65	77.37	85.36	86.71	37.44

Table 4.4 Axial forces in beams of edge frames in x-direction

Beam No.	Axial Load Capacity	Retrofitted With V-Pattern Brace	Retrofitted With Diamond Pattern Brace	Retrofitted With Cross Brace	Retrofitted With Infill Masonry Wall Type 1	Retrofitted With Infill Masonry WallType 2
	KN	KN	KN	KN	KN	KN
2001	500.00	61.57	54.91	56.97	20.12	172.07
2002	500.00	244.68	89.89	72.63	116.15	162.26
2011	500.00	9.33	16.55	8.63	14.55	97.69
2012	500.00	339.98	55.90	71.53	131.41	54.84
2021	500.00	10.39	15.15	14.63	16.65	86.24
2022	500.00	336.82	38.61	68.07	112.19	50.71
2061	500.00	11.57	13.33	15.39	23.83	54.85
2062	500.00	253.51	34.75	45.97	73.93	31.67
2071	500.00	11.30	14.85	15.22	24.71	47.30
2072	500.00	229.98	27.47	40.89	63.16	29.33
2111	500.00	14.26	16.44	19.89	34.71	15.69
2112	500.00	110.81	19.59	27.29	29.35	41.57
2121	500.00	15.05	25.21	34.22	62.66	16.15
2122	500.00	77.37	43.03	33.64	30.48	94.21
2131	500.00	77.65	77.37	74.24	63.55	43.49
2132	500.00	91.52	87.40	84.79	30.65	148.06

- The axial capacities of beams are calculated, by considering beams as columns and the minimum longitudinal reinforcement is taken at mid span i.e. 2-20mm diameter bars at both top and bottom face of beams.

Table 4.5 capacity and demand of axial forces for ground floor columns of edge frame in x- direction

Column No.	Existing Frame		Retrofitted With V-Pattern Brace	Retrofitted With Diamond Brace	Retrofitted With Cross Brace	Retrofitted With Infill Masonry Wall Type 1	Retrofitted With Infill Masonry Wall Type 2
	Capacity	Demands					
	KN		KN	KN	KN	KN	KN
1001	5145.50	2181.28	2000.79	2000.56	1972.48	1822.33	2822.48
1002	5145.50	2870.18	3984.21	3953.89	4417.10	3570.33	1987.47
1003	5145.50	2928.08	3479.96	3627.46	3602.75	1645.75	1554.81
1004	5145.50	2870.18	3984.21	3953.89	4417.10	3570.33	1987.47
1005	5145.50	2181.28	2000.79	2000.56	1972.48	1822.33	2822.48

Table 4.6 Factored B.M. at bottom node of ground floor columns in frames at edge in x-direction

Column No.	Existing Frame		Retrofitted With V-Pattern Brace	Retrofitted With Diamond Brace	Retrofitted With Cross Brace	Retrofitted With Infill Masonry Wall Type 1	Retrofitted With Infill Masonry WallType 2
	Capacity	Demand					
	KN×M	KN×M	KN×M	KN×M	KN×M	KN×M	KN×M
1001	365.70	**442.24**	360.51	337.64	296.41	105.99	45.65
1002	419.17	**450.24**	368.74	340.68	300.76	91.97	62.70
1003	421.37	**448.15**	315.95	355.56	290.85	75.17	37.54
1004	419.17	**450.24**	368.74	340.68	300.76	91.97	62.70
1005	365.70	**442.24**	360.51	337.64	296.41	105.99	45.65
Demands are more than capacity in existing frame before retrofitting as shown in bold figures.							

Table 4.7 Capacity and demands of shear force in ground floor columns at bottom section for frames at edge in x-direction

Column No.	Existing frame		Retrofitted With V-Pattern Brace	Retrofitted With Diamond Brace	Retrofitted With Cross Brace	Retrofitted With Infill Masonry Wall Type 1	Retrofitted With Infill Masonry Wall Type 2
	Capacity	Demand as per analysis					
	KN	KN	KN	KN	KN	KN	KN
1001	287.70	141.80	137.72	126.26	121.23	54.93	14.47
1002	287.70	149.33	141.54	126.57	121.00	39.16	50.23
1003	287.70	147.76	114.48	135.95	115.15	35.78	47.26
1004	287.70	149.33	141.54	126.57	121.00	39.16	50.23
1005	287.70	141.80	137.72	126.26	121.23	54.93	14.47

Table 4.8 Variation of bending moment along height of columns of edge frame in z-direction

Column No.	B.M. Capacity (Kn×M)	B.M. Demand As Per Revised Draft (Kn×M)	Retrofitted With Cross Brace (Kn×M)	Retrofitted With Infill Wall (Kn×M)	C/D Ratio Before Retrofitting	C/D Ratio After Retrofitting
	A BOTTOM	AT BOTTOM	AT BOTTOM	AT BOTTOM		
1001	359.52	**385.94**	295.93	98.49	**0.932**	1.383
1011	364.30	**377.55**	140.73	84.36	**0.965**	2.588
1021	370.51	190.63	122.22	95.99	1.944	3.030
1031	377.01	122.83	125.40	101.03	3.069	3.006
1041	383.37	102.36	126.96	104.48	3.745	3.019
1051	389.42	96.91	129.72	106.73	4.018	3.002
1061	390.14	90.14	132.47	107.82	4.328	2.945
1071	395.16	83.33	134.17	107.89	4.742	2.945
1081	399.60	76.84	134.95	107.11	4.742	2.961
1091	403.70	73.79	134.87	105.64	5.471	2.993
1101	407.21	74.64	133.72	103.64	5.455	3.045
1111	409.95	75.08	130.20	101.18	5.460	3.148
1121	412.07	77.22	128.87	100.40	5.336	3.198
1131	414.40	122.83	172.93	102.78	3.374	2.396

- The bold figures show unsafe bending moment demands and C/D ratio.

Table 4.9 Capacity and demands of axial forces in beams of frames at edge in z-direction

Beam No.	Axial Load Capacity	Retrofitted With Cross Pattern Brace	Retrofitted With Infill wall Type 1
	KN	KN	KN
2001	500.00	49.10	11.19
2002	500.00	72.31	102.62
2011	500.00	6.81	14.64
2012	500.00	5178	95.67
2021	500.00	12.42	16.04
2022	500.00	56.93	87.18
2061	500.00	12.60	18.51
2062	500.00	36.83	56.50
2071	500.00	12.06	18.55
2072	500.00	31.78	48.82
2111	500.00	12.51	21.27
2112	500.00	13.31	17.83
2121	500.00	30.48	51.03
2122	500.00	24.60	11.63
2131	500.00	69.85	72.75
2132	500.00	74.99	6.69

- The axial load carrying capacity of beams are calculated, considering the beams as columns and minimum longitudinal steel at mid span i.e. two bars of 20 mm diameter both at top and bottom faces of beams. Thus beams are safe against axial forces.

Table 4.10 Joint displacements at end columns of edge frame in x-direction

Joint No.	Existing Frame	Retrofitted With V-Pattern Brace	Retrofitted With Diamond Pattern Brace	Retrofitted With Cross Brace	Retrofitted With Infill Wall Type 1	Retrofitted With Infill Wall Type 2
	CM	CM	CM	CM	CM	CM
1005	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1015	0.3370	0.2507	0.2378	0.1975	0.0818	0.0404
1025	1.0004	0.6585	0.6388	0.4957	0.1954	0.1236
1035	1.7596	1.0799	1.0468	0.7989	0.3267	0.2486
1045	2.5232	1.4925	1.4489	1.1023	0.4768	0.4063
1055	3.2555	1.8894	1.8341	1.4025	0.6424	0.5916
1065	3.9415	2.2665	2.2034	1.6959	0.8198	0.7994
1075	4.5742	2.6203	2.5494	1.9789	1.0053	1.0249
1085	5.1493	2.9480	2.8723	2.2484	1.1958	1.2632
1095	5.6627	3.2462	3.1668	2.5013	1.3881	1.5102
1105	6.1097	3.5116	3.4308	2.7342	1.5795	1.7618
1115	6.4848	3.7403	3.6599	2.9438	1.7672	2.0145
1125	6.7831	3.9289	3.8507	3.1276	1.9495	2.2650
1135	7.0019	4.0741	4.0122	3.2829	2.1272	2.5101
1145	7.1285	4.1530	4.0844	3.3799	2.2736	2.7490

Table 4.11 Joint displacements at end columns of frames in z-direction

Joint No.	Edge Frame			Middle Frame		
	Existing Frame	Retrofitted With Cross Brace	Retrofitted With Infill Wall Type 1	Existing Frame	Retrofitted With Cross Brace	Retrofitted With Infill Wall Type 1
	CM	CM	CM	CM	CM	CM
1004	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1014	0.3212	0.2055	0.0614	0.4190	0.2747	0.0835
1024	0.9544	0.5431	0.1745	1.2418	0.7202	0.2322
1034	1.6794	0.9060	0.3312	2.1829	1.1944	0.4371
1044	2.4099	1.2779	0.5249	3.1305	1.6757	0.6898
1054	3.1119	1.6523	0.7479	4.0423	2.1559	0.9800
1064	3.7711	2.0234	0.9936	4.9008	2.6290	1.2999
1074	4.3811	2.3867	1.2557	5.6970	3.0909	1.6395
1084	4.9377	2.7380	1.5288	6.4244	3.5378	1.9942
1094	5.4374	3.0733	1.8084	7.0765	3.9659	2.3573
1104	5.8755	3.3884	2.0902	7.6471	4.3704	2.7237
1114	6.2465	3.6787	2.3709	8.1291	4.7459	3.0892
1124	6.5449	3.9403	2.6480	8.5168	5.0865	3.4505
1134	6.7689	4.1698	2.9228	8.8088	5.3874	3.8098
1144	6.9113	4.3422	3.1742	8.9908	5.6092	4.1364

Table 4.12 Storey drifts in edge frames in x-direction

Floor Level	Existing	Retrofitted With V-Pattern Brace	Retrofitted With Diamond Pattern Brace	Retrofitted With Cross Brace Pattern	Retrofitted With Infill Wall Type 1	Retrofitted With Infill WallType 2
	CM	CM	CM	CM	CM	CM
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	0.3370	0.2507	0.2378	0.1975	0.0818	0.0404
2	0.6634	0.4078	0.4010	0.2982	0.1136	0.0832
3	0.7592	0.4214	0.4080	0.3032	0.1313	0.1250
4	0.7636	0.4126	0.4021	0.3034	0.1501	0.1577
5	0.7323	0.3969	0.3852	0.3002	0.1656	0.1880
6	0.6860	0.3771	0.3693	0.2934	0.1774	0.2078
7	0.6327	0.3538	0.3460	0.2830	0.1855	0.2255
8	0.5751	0.3277	0.3229	0.2695	0.1905	0.2383
9	0.5134	0.2982	0.2945	0.2529	0.1923	0.2470
10	0.4470	0.2656	0.2640	0.2329	0.1914	0.2516
11	0.3751	0.2287	0.2291	0.2096	0.1877	0.2527
12	0.2983	0.1886	0.1908	0.1838	0.1823	0.2505
13	0.2188	0.1452	0.1615	0.1553	0.1777	0.2451
14	0.1266	0.0789	0.0722	0.0970	0.1464	0.2389

Table 4.13 Story drifts in frames in z-direction

Floor Level	Edge Frame			Middle Frame		
	Existing Frame	Retrofitted With Cross Brace	Retrofitted With Complete Infill	Existing Frame	Retrofitted With Cross Brace	Retrofitted With Complete Infill
	CM	CM	CM	CM	CM	CM
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	0.3212	0.2055	0.0614	0.4190	0.2747	0.0835
2	0.6332	0.3376	0.1131	0.8228	0.4455	0.1487
3	0.7250	0.3629	0.1567	0.9411	0.4742	0.2049
4	0.7301	0.3719	0.1937	0.9476	0.4813	0.2587
5	0.7024	0.3744	0.2230	0.9118	0.4802	0.2902
6	0.6592	0.3711	0.2457	0.8585	0.4731	0.3199
7	0.6100	0.3633	0.2621	0.7962	0.4619	0.3394
8	0.5566	0.3513	0.2731	0.7274	0.4469	0.3547
9	0.4997	0.3353	0.2796	0.6521	0.4281	0.3631
10	0.4381	0.3151	0.2818	0.5706	0.4045	0.3664
11	0.3710	0.2903	0.2807	0.4820	0.3755	0.3655
12	0.2984	0.2616	0.2771	0.3877	0.3406	0.3613
13	0.2240	0.2294	0.2748	0.2920	0.3009	0.3593
14	0.1424	0.1724	0.2514	0.1820	0.2218	0.3266

Table 4.14 Variation in axial forces in beams of frames in z-direction

Beam No.	Edge Frame			Middle Frame		
	Existing Frame	Retrofitted With Cross Brace	Retrofitted With Infill Wall Type 1	Existing Frame	Retrofitted With Cross Brace	Retrofitted With Infill Wall Type 1
	KN	KN	KN	KN	KN	KN
2001	26.58	49.10	11.19	38.82	68.30	17.69
2002	13.15	72.31	102.62	22.96	126.20	179.06
2011	6.57	16.81	14.64	9.35	10.35	22.30
2012	3.97	51.78	95.67	6.93	112.56	166.89
2021	3.15	12.42	16.04	5.34	18.15	24.65
2022	2.84	56.93	87.18	4.96	99.31	152.07
2061	1.98	12.60	18.51	3.55	17.73	27.11
2062	2.04	36.83	56.50	3.55	64.19	98.45
2071	1.88	12.06	18.55	3.35	16.55	26.57
2072	1.75	31.78	48.82	3.05	55.35	85.05
2111	2.39	12.51	21.27	4.01	19.77	27.73
2112	2.95	13.31	17.83	5.07	23.05	30.93
2121	18.18	30.48	51.03	29.30	44.06	70.60
2122	16.49	24.60	11.63	27.73	41.76	19.82
2131	64.09	69.85	72.75	110.69	111.09	109.97
2132	80.49	74.99	6.69	139.41	129.90	11.35

Table 4.15 Story shear forces for existing building frames

Floor Level	X-Direction		Z-Direction	
	Edge Frame (Kn)	Middle Frame (Kn)	Edge Frame (Kn)	Middle Frame (Kn)
Ground	737.93	1009.31	570.12	766.82
1	730.51	983.14	558.62	752.60
2	725.81	981.56	557.62	744.72
3	700.66	953.03	539.72	730.46
4	672.97	922.08	518.94	707.62
5	644.20	889.14	497.30	683.18
6	614.54	852.12	475.28	656.08
7	583.25	809.80	452.40	625.30
8	548.86	762.17	427.08	590.70
9	509.20	565.06	397.60	552.04
10	461.03	649.29	361.36	507.46
11	400.52	577.95	314.28	451.10
12	333.03	499.97	258.98	385.62
13	258.29	411.43	213.42	337.28

Table 4.16 End shear forces at section I in beams of existing and retrofitted edge frame in x-direction

Beam No.	Before Retrofitting		After Retrofitting (Kn)				
	Capacity (Kn)	Demand (Kn)	V-Pattern	Diamond Pattern	Cross Pattern	Infill Wall Type 1	Infill Wall Type 2
2001	265.58	156.54	137.97	140.68	134.33	134.16	134.51
2002	265.58	155.19	137.62	136.84	133.63	133.79	137.82
2011	269.58	174.17	132.44	147.74	138.64	134.44	141.84
2012	269.58	171.32	135.75	139.23	133.43	133.65	134.20
2021	269.58	178.73	132.02	138.36	141.52	134.67	133.82
2022	269.58	175.07	137.40	150.30	133.39	133.59	134.02
2061	269.58	169.51	132.95	133.43	143.84	135.14	146.38
2062	269.58	163.83	145.17	149.41	133.40	133.57	133.86
2071	269.58	165.67	132.17	133.49	144.28	135.23	146.23
2072	269.58	159.61	147.26	147.04	133.39	133.57	134.12
2111	256.48	142.46	132.07	133.67	146.97	135.35	152.90
2112	269.98	135.23	157.35	146.88	133.38	133.55	133.63
2121	256.48	133.84	131.83	133.40	146.89	135.10	151.65
2122	269.98	126.70	148.03	123.90	123.51	133.57	133.36
2131	256.48	115.19	111.49	133.34	132.56	124.70	123.85
2132	269.98	110.18	138.05	118.23	115.07	114.67	114.63

Table 4.17 Bending moment capacity demand ratios for retrofitted and existing edge frame in x-direction

Beam No.	Before Retrofitting		After Retrofitting									
			V-Pattern Brace		Diamond Pattern Brace		Cross Brace Pattern		Infill Masonry Wall Type 1		Infill Masonry Wall Type 2	
	NODE I	NODE J	I	J	I	J	I	J	I	J	I	J
2001	**0.731**	**0.749**	**0.91**	**0.93**	**0.90**	**0.93**	**0.99**	1.03	1.34	1.13	1.30	1.35
2002	**0.744**	**0.709**	**0.97**	1.00	**0.95**	**0.95**	1.08	1.08	1.39	1.19	1.26	1.28
2011	**0.716**	**0.757**	**0.99**	1.06	**0.96**	1.03	1.10	1.18	1.46	1.24	1.72	1.39
2012	**0.740**	**0.741**	1.12	1.10	1.08	1.09	1.29	1.28	1.66	1.42	1.48	1.30
2021	**0.686**	**0.746**	**0.96**	1.07	1.05	1.05	1.05	1.18	1.35	1.16	1.73	1.35
2022	**0.713**	**0.716**	1.14	1.11	1.12	1.09	1.31	1.30	1.67	1.38	1.31	1.38
2061	**0.751**	**0.908**	1.29	1.20	1.18	1.18	1.03	1.24	1.27	1.10	1.00	1.27
2062	**0.799**	**0.808**	**0.96**	1.25	1.25	1.25	1.46	1.45	1.67	1.04	1.72	1.11
2071	**0.781**	**0.781**	1.35	1.34	1.23	1.00	1.01	1.02	1.16	1.42	1.00	1.38
2072	**0.673**	**0.681**	1.08	1.23	1.01	1.05	1.21	1.20	1.12	1.03	1.39	1.09
2111	**0.833**	1.068	**0.84**	1.18	1.05	1.15	**0.79**	1.10	1.05	1.01	**0.74**	1.37
2112	**0.817**	**0.837**	1.38	1.17	1.20	1.18	1.19	1.19	1.35	1.42	1.23	**0.73**
2121	**0.947**	1.283	**0.96**	1.31	**0.95**	1.28	**0.90**	1.18	**0.91**	**0.70**	**0.74**	**0.95**
2122	**0.942**	**0.961**	1.23	1.17	1.19	1.19	1.20	1.19	1.19	1.21	1.39	1.16
2131	1.210	1.535	1.17	1.46	1.16	1.44	1.09	1.30	1.06	**0.74**	1.69	1.01
2132	1.177	1.218	1.48	1.35	1.44	1.38	1.45	1.38	1.38	1.09	**0.72**	**0.85**

- Bold figures represent unsafe C/D ratios. The sections of member showing C/D ratio <1 need Individual retrofitting.

Table 4.18 Bending moment capacity demand ratio before and after retrofitting of edge frame in z-direction

Beam No.	Before Retrofitting		After Retrofitting			
	Existing Frame		Retrofitted With Cross Brace Pattern		Retrofitted With Infill Wall Type 1	
	NODE I	NODE J	NODE I	NODE J	NODE I	NODE J
2001	**0.746**	**0.699**	1.000	**0.911**	1.594	1.372
2002	**0.680**	**0.680**	1.015	1.015	1.533	1.533
2011	**0.764**	**0.683**	1.102	1.002	1.682	1.381
2012	**0.673**	**0.667**	1.201	1.201	1.829	1.829
2021	**0.730**	**0.673**	1.041	1.101	1.489	1.227
2022	**0.644**	**0.643**	1.242	1.242	1.829	1.829
2061	**0.808**	**0.830**	**0.959**	1.008	1.192	1.010
2062	**0.729**	**0.727**	1.485	1.485	1.829	1.829
2071	**0.843**	**0.892**	1.023	1.036	1.172	**0.971**
2072	**0.766**	**0.765**	1.545	1.552	**0.841**	1.829
2111	**0.821**	**0.958**	**0.769**	**0.856**	1.534	1.646
2112	**0.921**	**0.721**	1.558	1.223	**0.845**	1.205
2121	**0.948**	1.201	**0.819**	**0.945**	1.534	**0.655**
2122	1.082	**0.847**	1.559	1.225	**0.840**	1.215
2131	1.227	1.506	1.000	1.062	1.000	**0.704**
2132	1.368	1.071	1.852	1.379	1.777	1.395

- Bold figures represent unsafe sections of member, as capacity demand ratios are less than unity. The members showing c/d ratio <1 need individual retrofitting.

Table 4.19 Capacity demand ratio for bending moment at mid spans of beams of existing and retrofitted edge frames in x-direction

Column No.	Before Retrofitting	After Retrofitting				
		V-Pattern Brace	Diamond Pattern Brace	Cross Pattern Brace	Infill Masonry Wall Type 1	Infill Masonry Wall Type 2
2001	**0.771**	2.318	2.318	2.319	2.310	2.322
2002	**0.641**	1.939	1.941	1.942	1.954	1.951
2011	**0.626**	2.329	2.330	2.331	2.312	2.312
2012	**0.636**	2.376	2.377	2.375	2.402	2.408
2021	**0.604**	2.329	2.329	2.329	2.312	2.302
2022	**0.609**	2.376	2.375	2.375	2.401	2.490
2061	**0.715**	2.328	2.329	2.329	2.312	2.315
2062	**0.701**	2.369	2.368	2.368	2.401	2.413
2071	**0.599**	1.828	1.829	1.829	1.815	1.822
2072	**0.583**	1.858	1.858	1.873	1.886	1.881
2111	**0.978**	1.892	1.897	1.892	1.878	1.893
2112	**0.621**	1.295	1.278	1.295	1.314	1.340
2121	**0.819**	1.896	1.897	1.896	1.882	1.872
2122	**0.754**	1.290	1.289	1.616	1.315	1.412
2131	1.032	2.112	2.115	2.114	2.101	2.121
2132	**0.974**	1.484	1.483	1.483	1.550	1.582

- The bold figures show unsafe sections.

Table 4.20 Comparison of methods used for retrofitting on the basis of performance and materials required for edge frame in x-direction

S. No.	Particulars	Existing	V-pattern	Diamond pattern	Cross pattern	Infill Wall Type 1	Infill Wall Type 2
1	Fundamental time period (Sec.)	1.802	1.080	1.086	0.899	0.490	0.625
2	Base shear (KN) (% increase w.r.t. existing frame)	449.31	682.43 (51.18)	688.32 (53.19)	788.60 (75.51)	1240.73 (176.0)	899.58 (100.0)
3	% Decrease in roof level joint displacements w.r.t. existing frame	-	40.17	42.70	52.59	68.10	61.44
4	Story drifts at ground floor(cm)	0.3370	0.2507	0.2378	0.1975	0.0818	0.0404
	Story drifts at roof level (cm)	0.1266	0.0781	0.0722	0.0970	0.1464	0.2389
5	Number of beams required individual strengthening	-	16	120	6	6	10
6	Weight of steel used in braces (KN)	-	7996.8	7996.8	10012.8	-	-
7	Number of steel connection plates(500mm×500mm×10 mm) at the end of beams	-	112	60	120	-	-
8	Number of steel plates(300mm× 300mm×10 mm) required at intersection	-	-	-	28	-	-
9	Number of threaded rod bolt 25mm diameter 600mm	-	252	180	360	-	-
10	Number of connecting T-sections ISST 250@ 37.5 Kg/m at the end of beams	-	56	56	112	-	-
11	Volume of block masonry for infill walls	-	-	-	-	90.48	90.48
12							

FIGURES

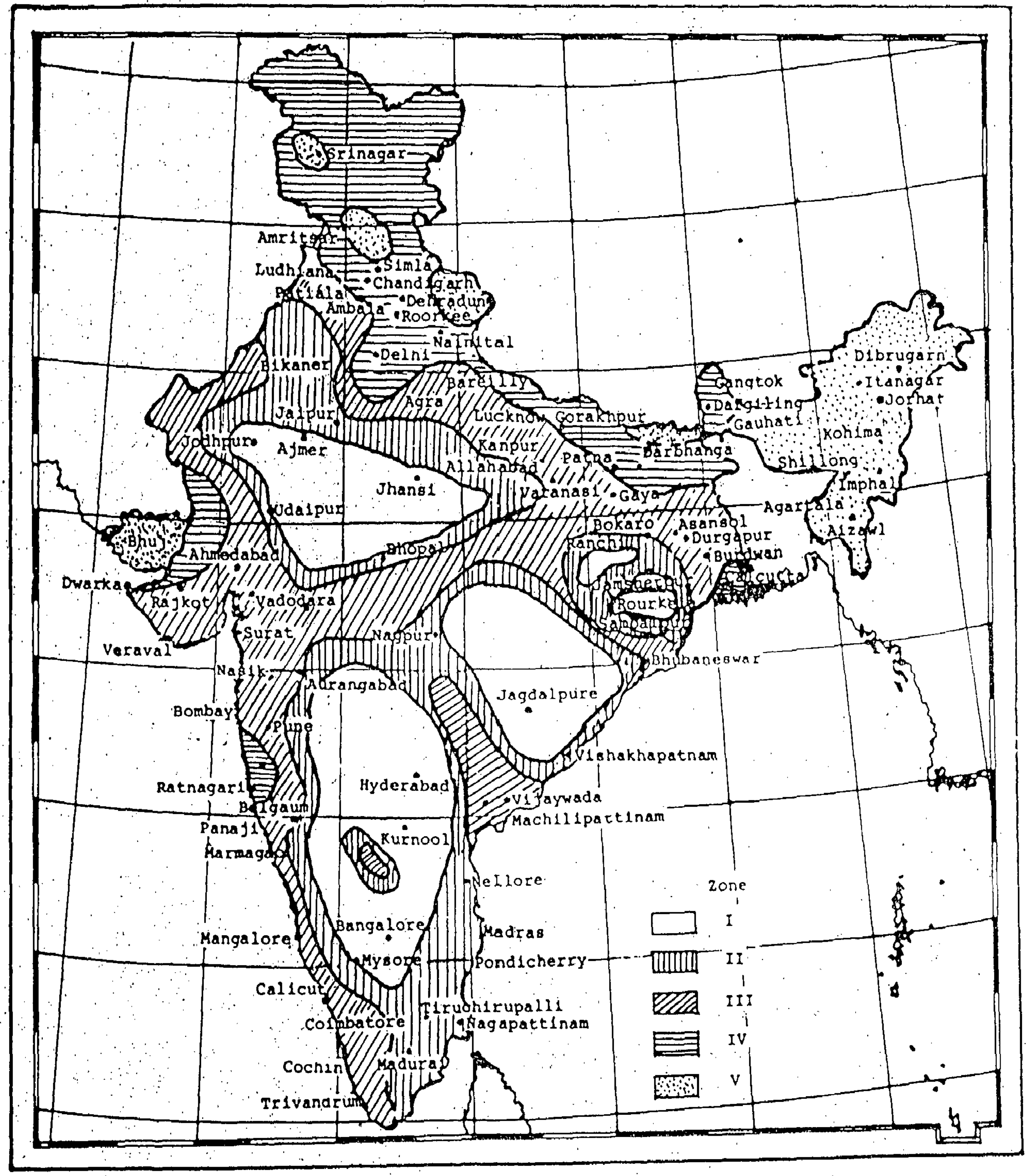


Fig. 2.1 Seismic zone map of India

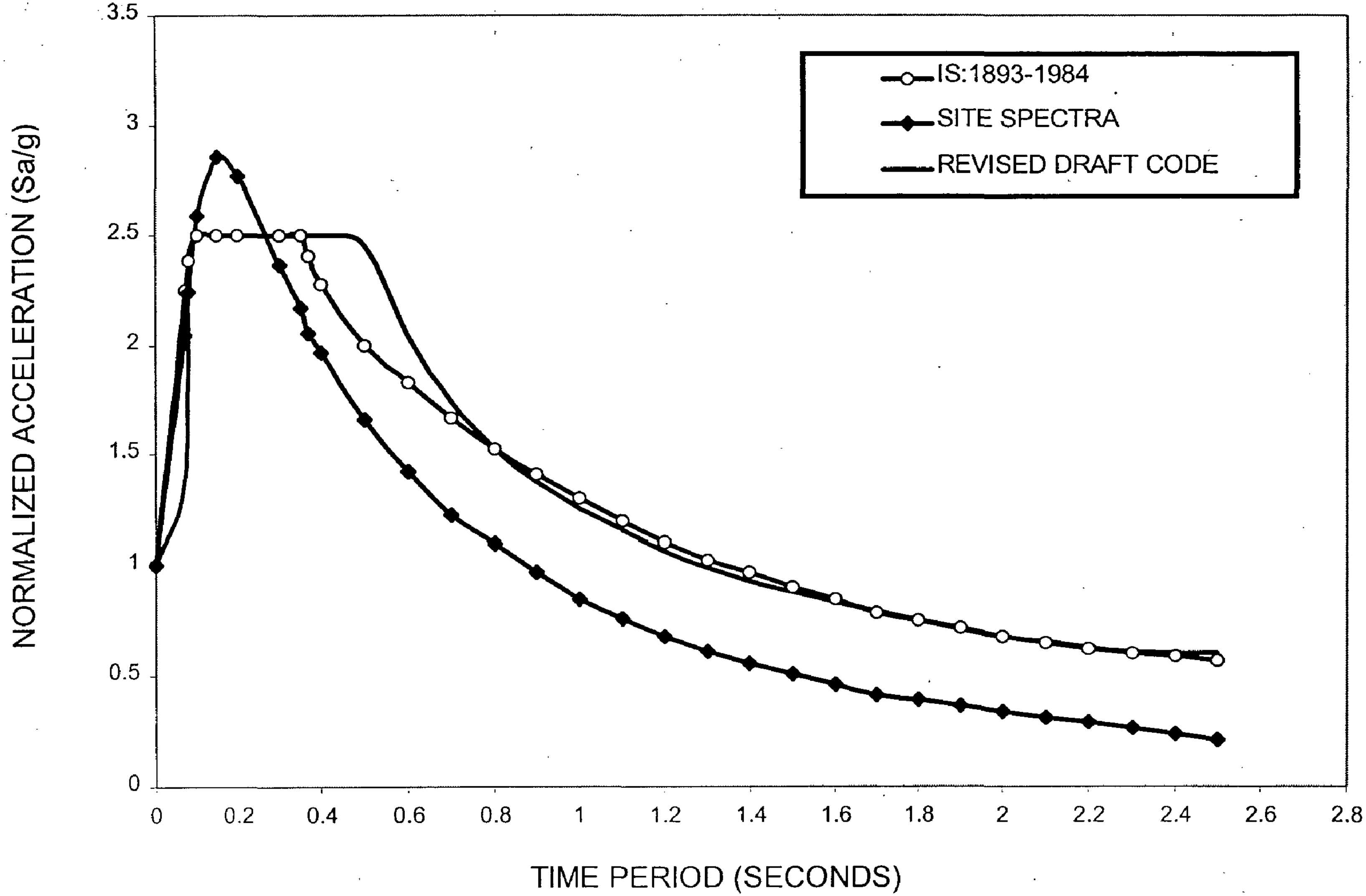


Fig. 2.2 Response spectra for soil site for 5% damping

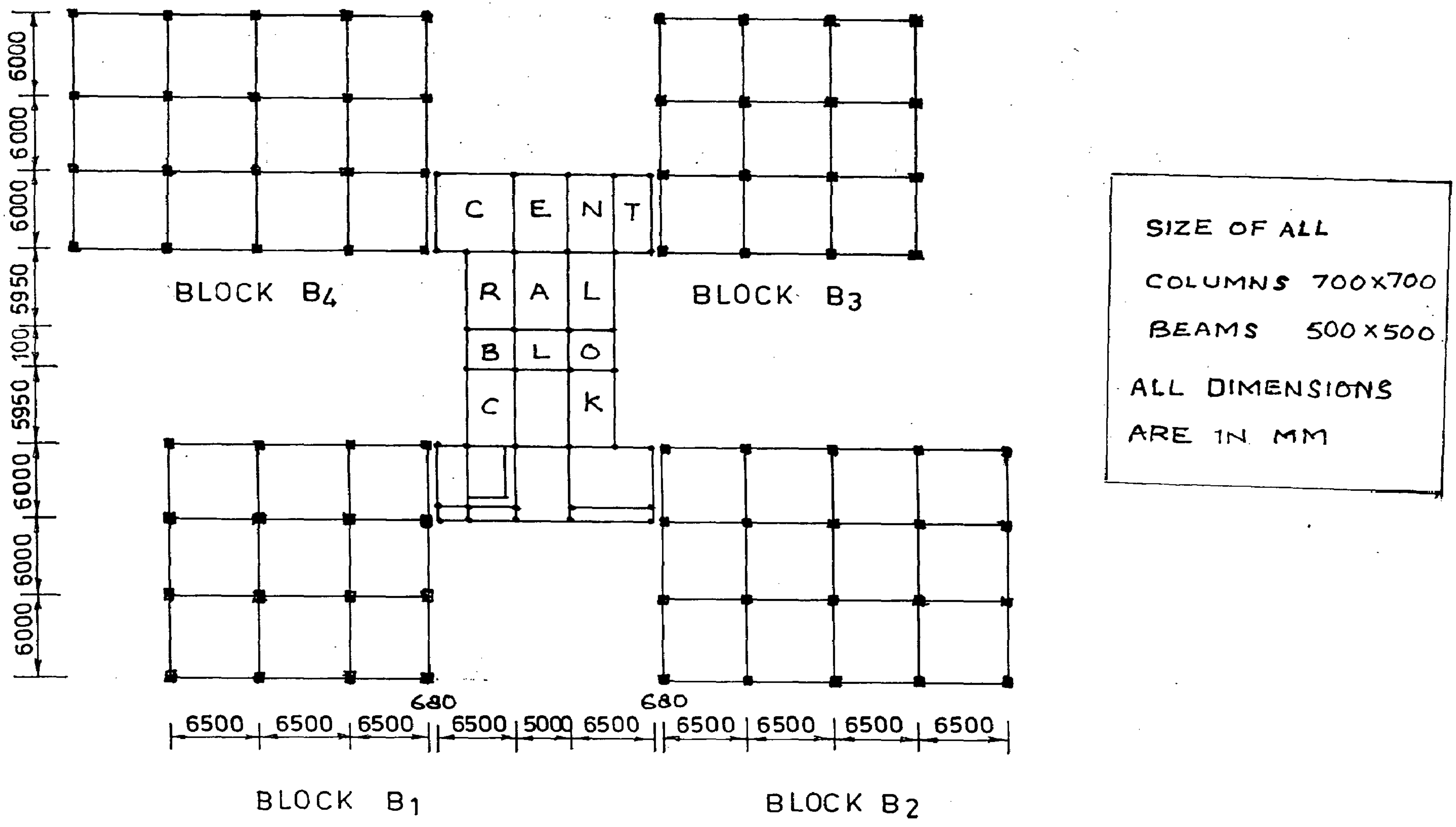


Fig. 3.1 Typical plan of 14-storied R.C. framed building in New Delhi

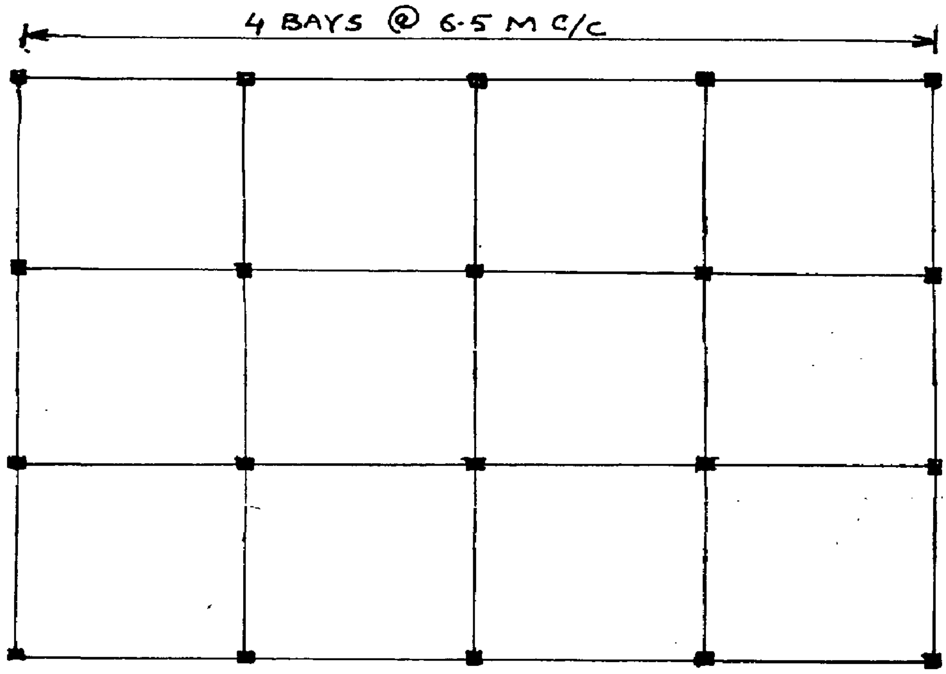
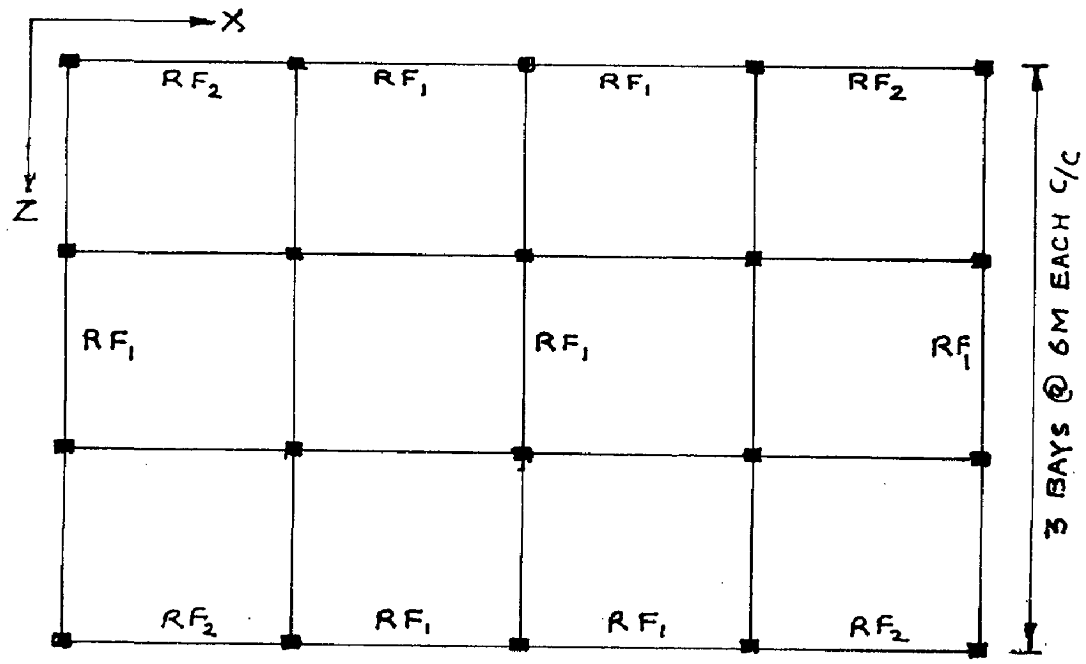


Fig. 3.2(a) Typical floor plan of block B_2 before retrofitting



RF1 LOCATIONS FOR STEEL BRACING AND INFILL MASONRY WALL TYPE 1

RF2 LOCATION FOR INFILL MASONRY WALL TYPE 2

Fig. 3.2(b) Typical floor plan of block B_2 after retrofitting

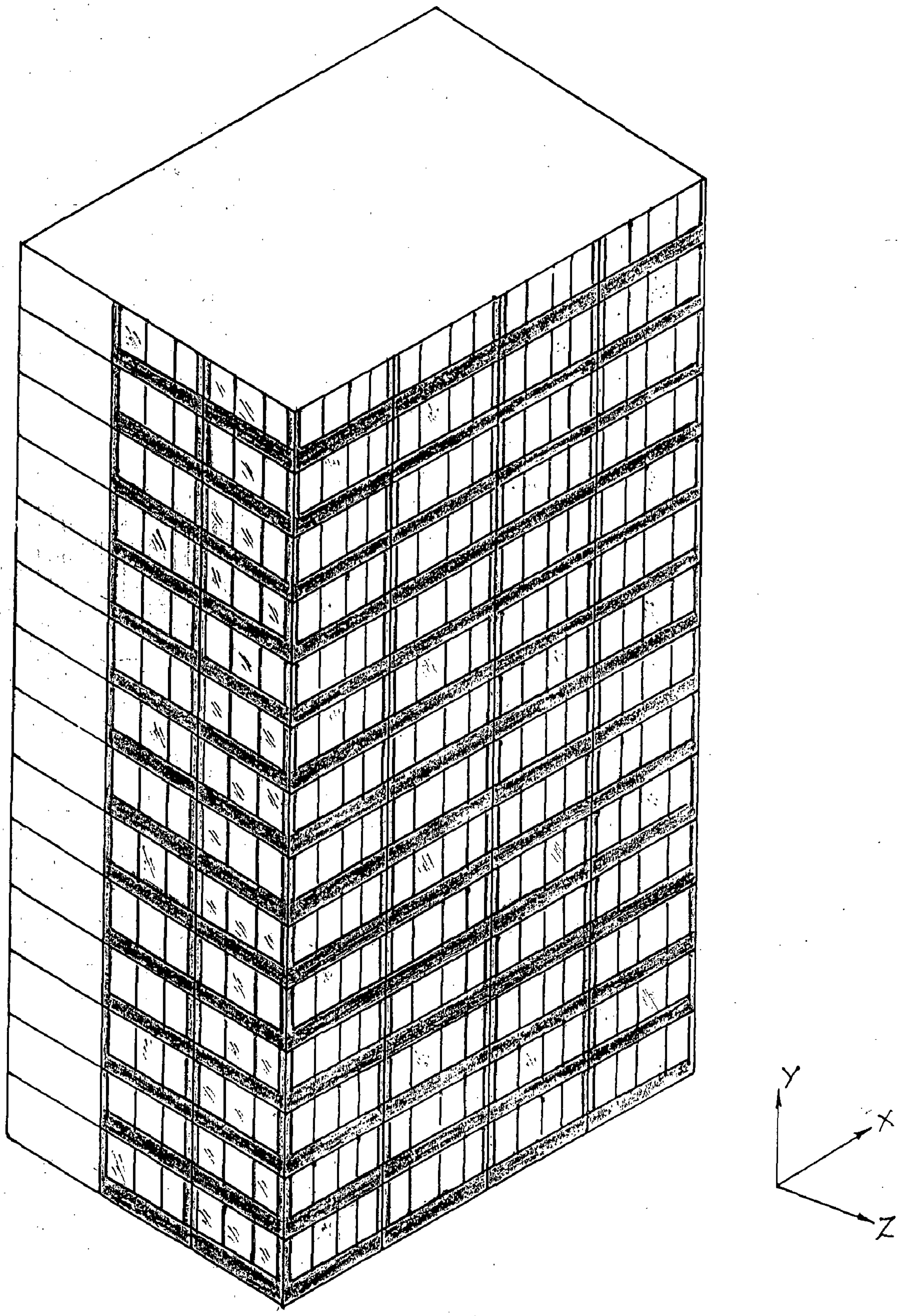


Fig. 3.3 Three dimensional view of existing block B_2 before retrofitting

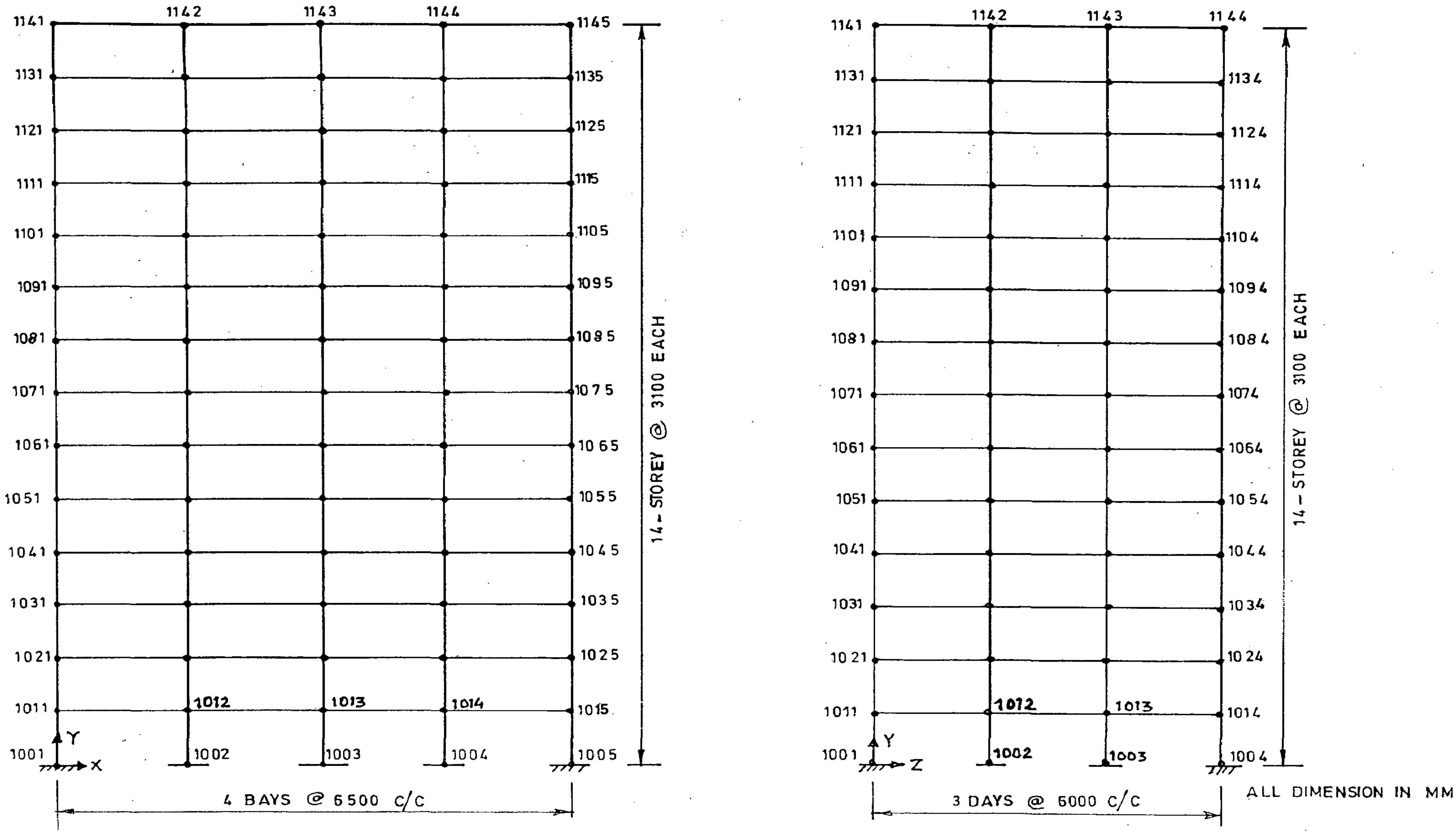


Fig. 3.4 Joint numbering in frames

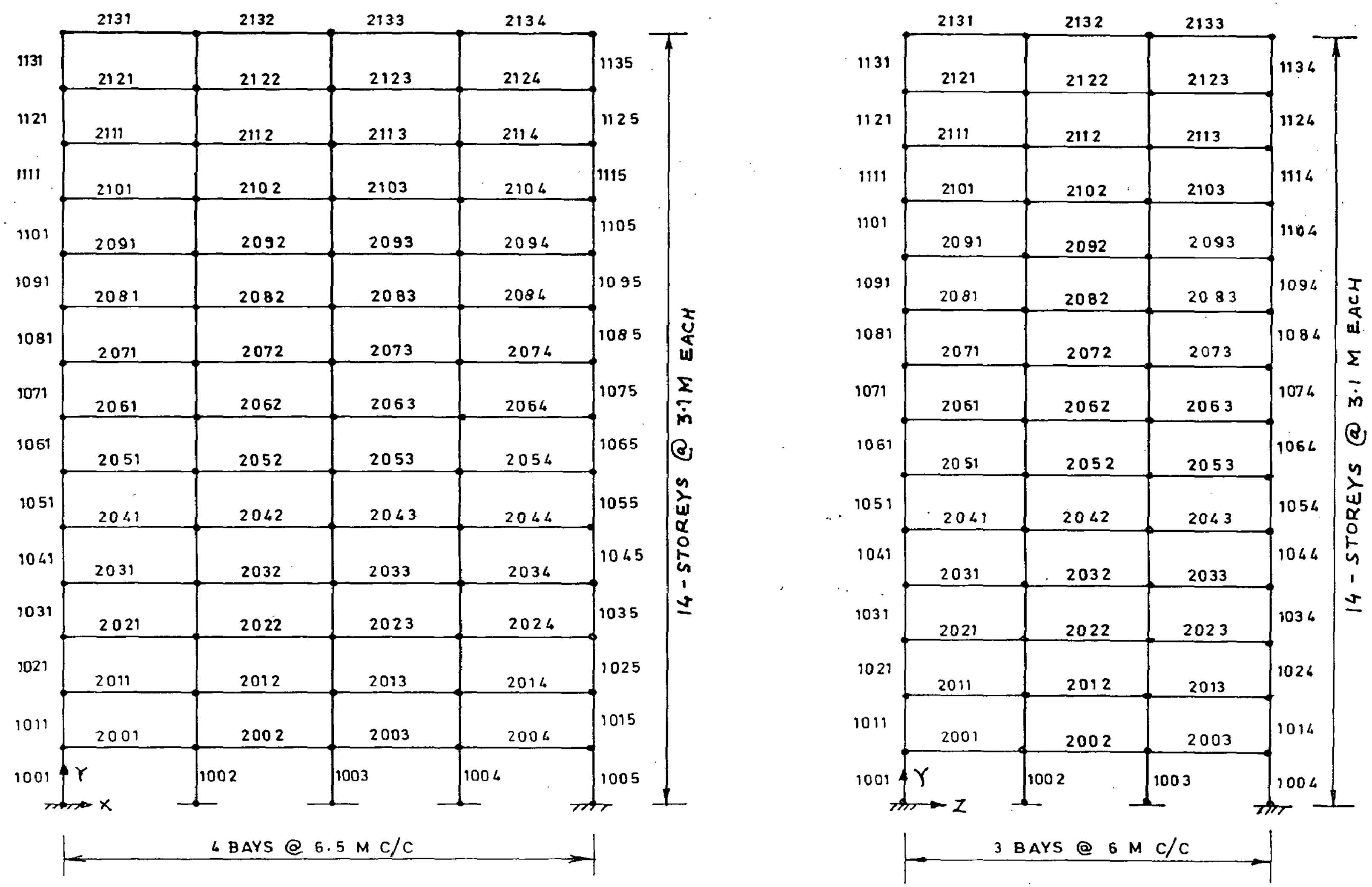


Fig. 3.5 Member numbering in frames

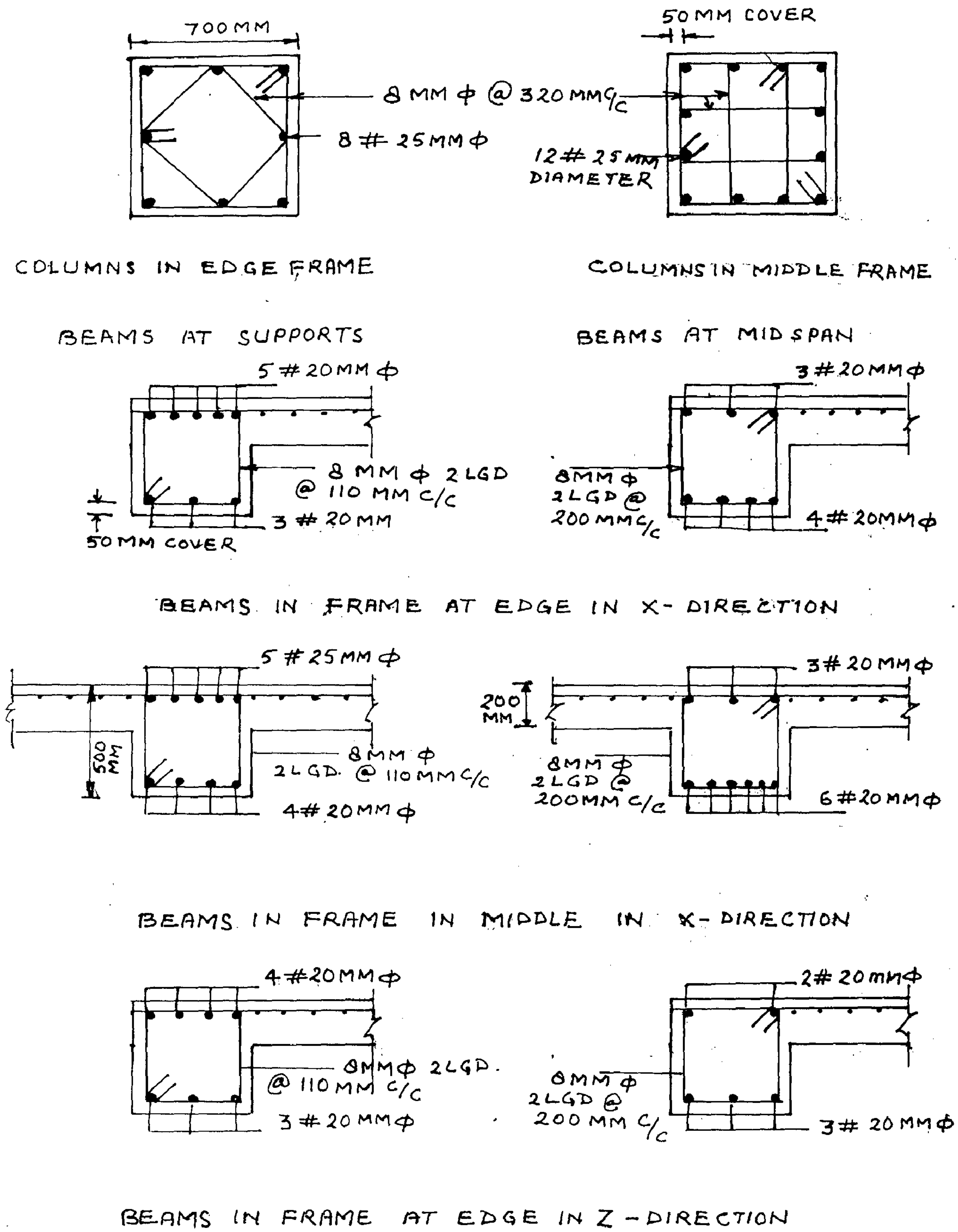


Fig. 3.6 Details of reinforcement in ground floor beams and columns

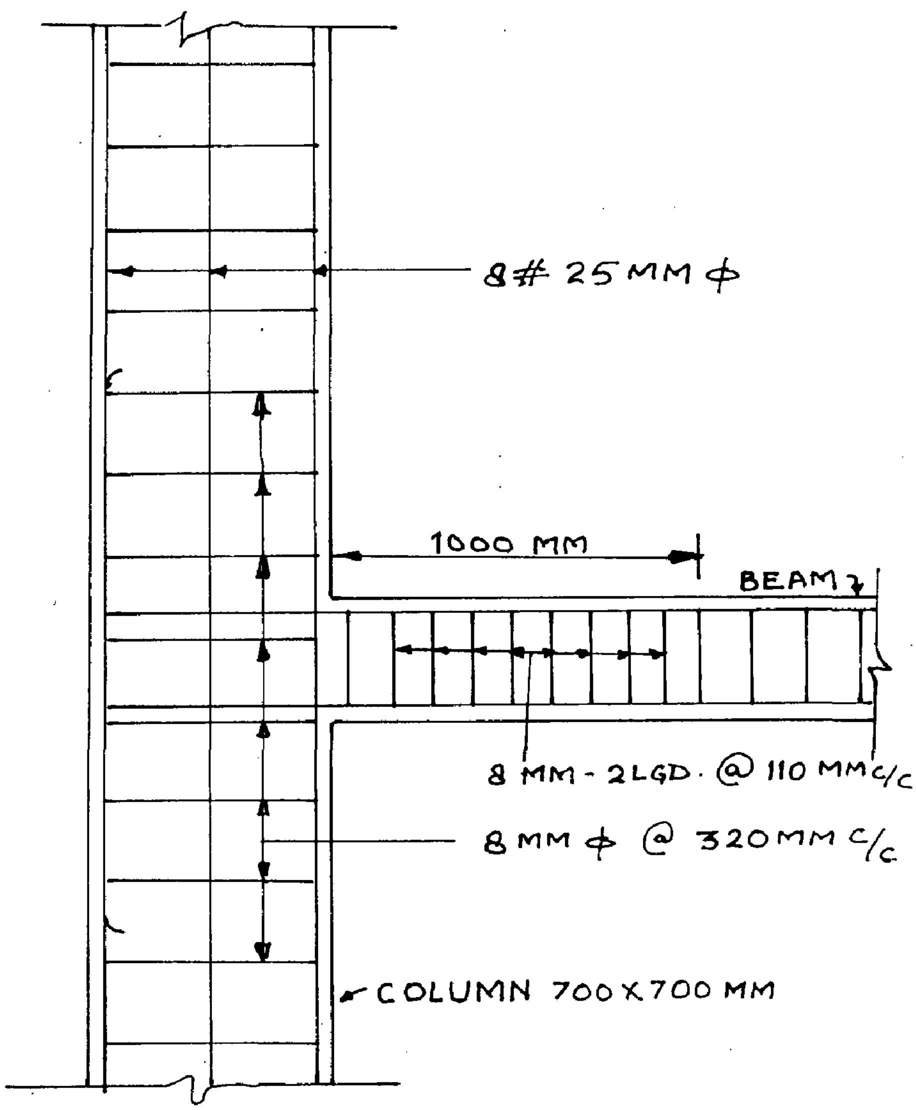


Fig. 3.7 Details of reinforcement in ground floor beam-column joint

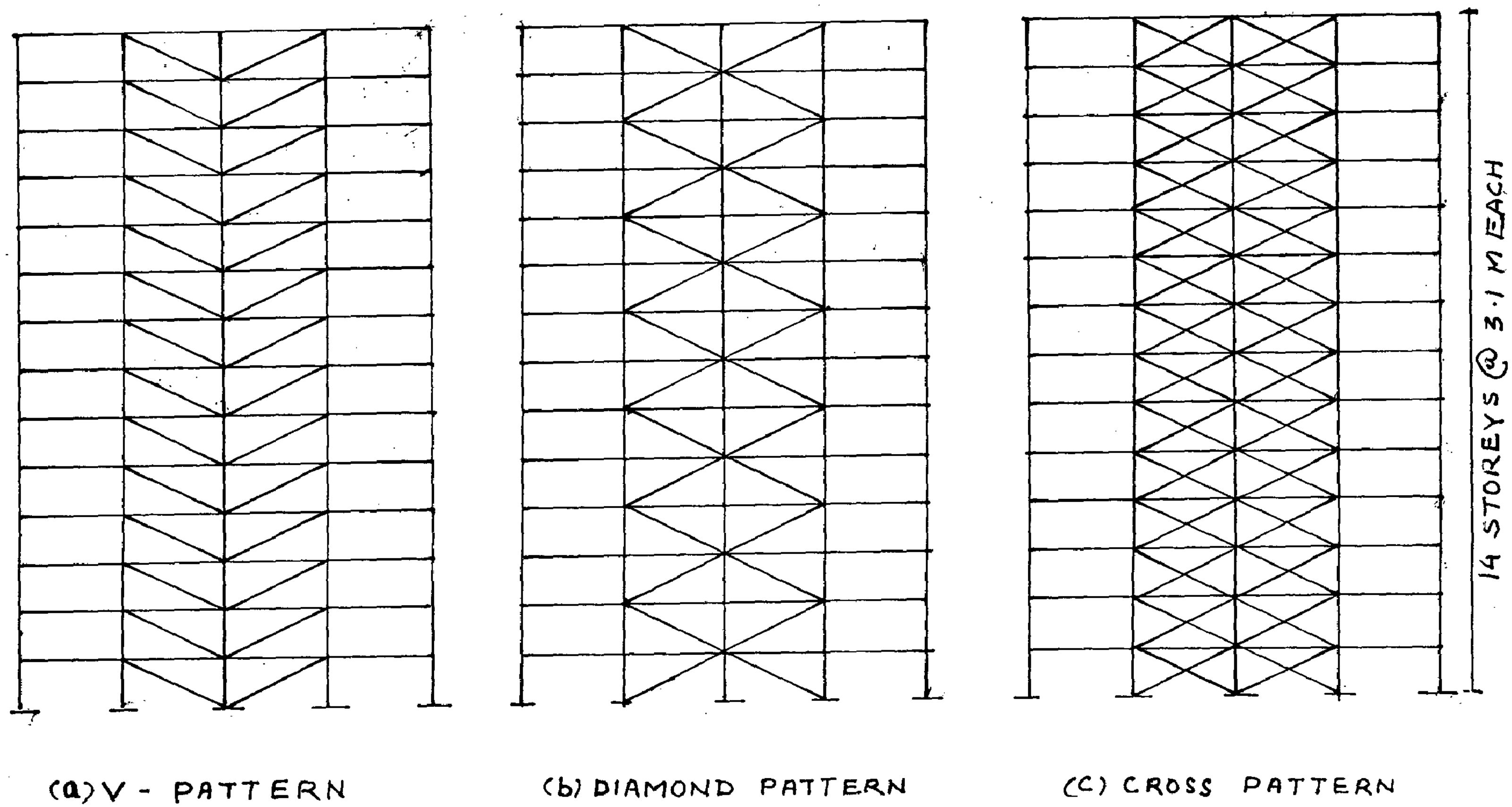
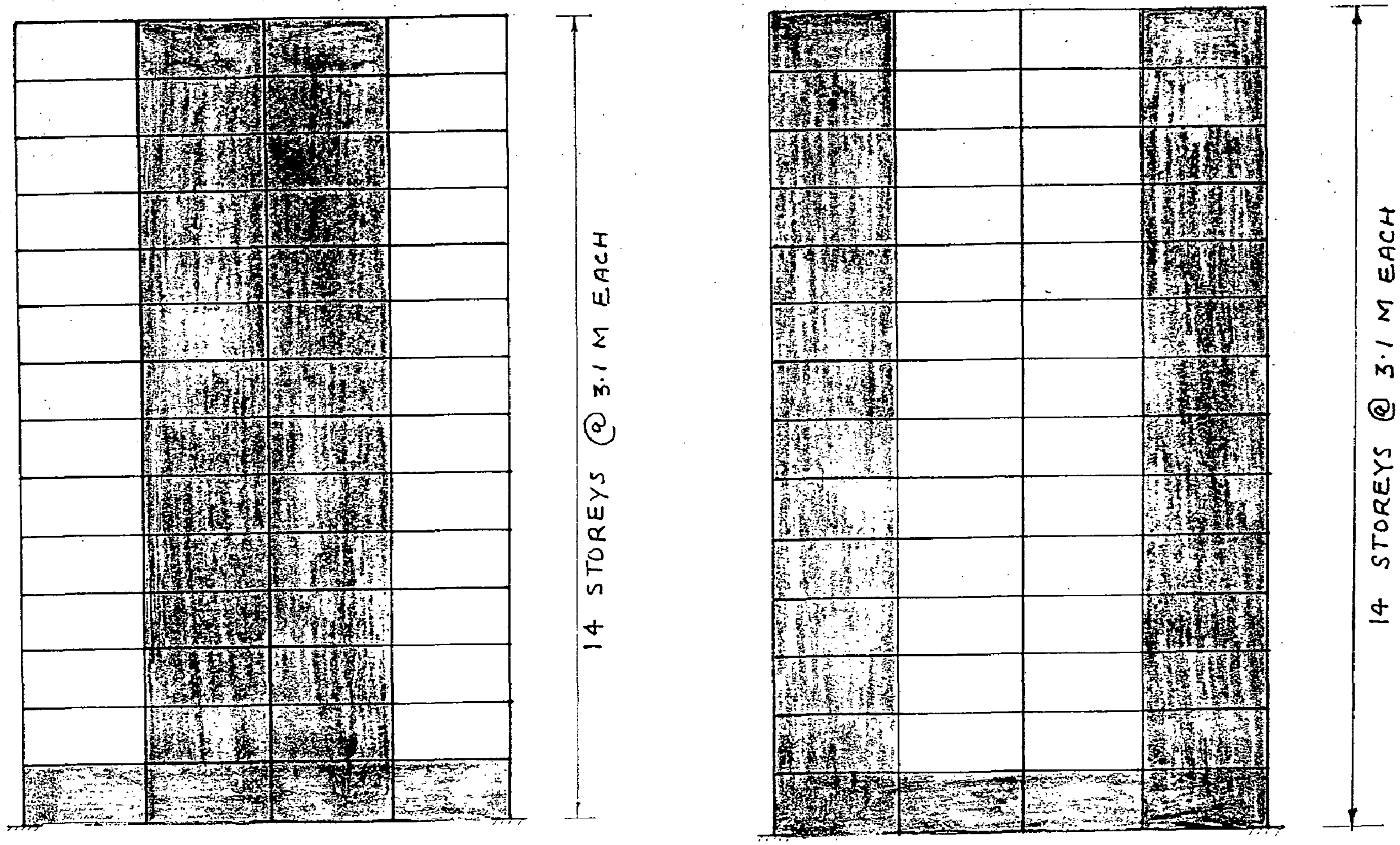


Fig. 4.1 Steel bracing patterns used in retrofitting for frame in x-direction



(a) INFILL MASONRY WALL TYPE1

(b) INFILL MASONRY WALL TYPE2

Fig. 4.2 Infill masonry wall types used for frame in x-direction

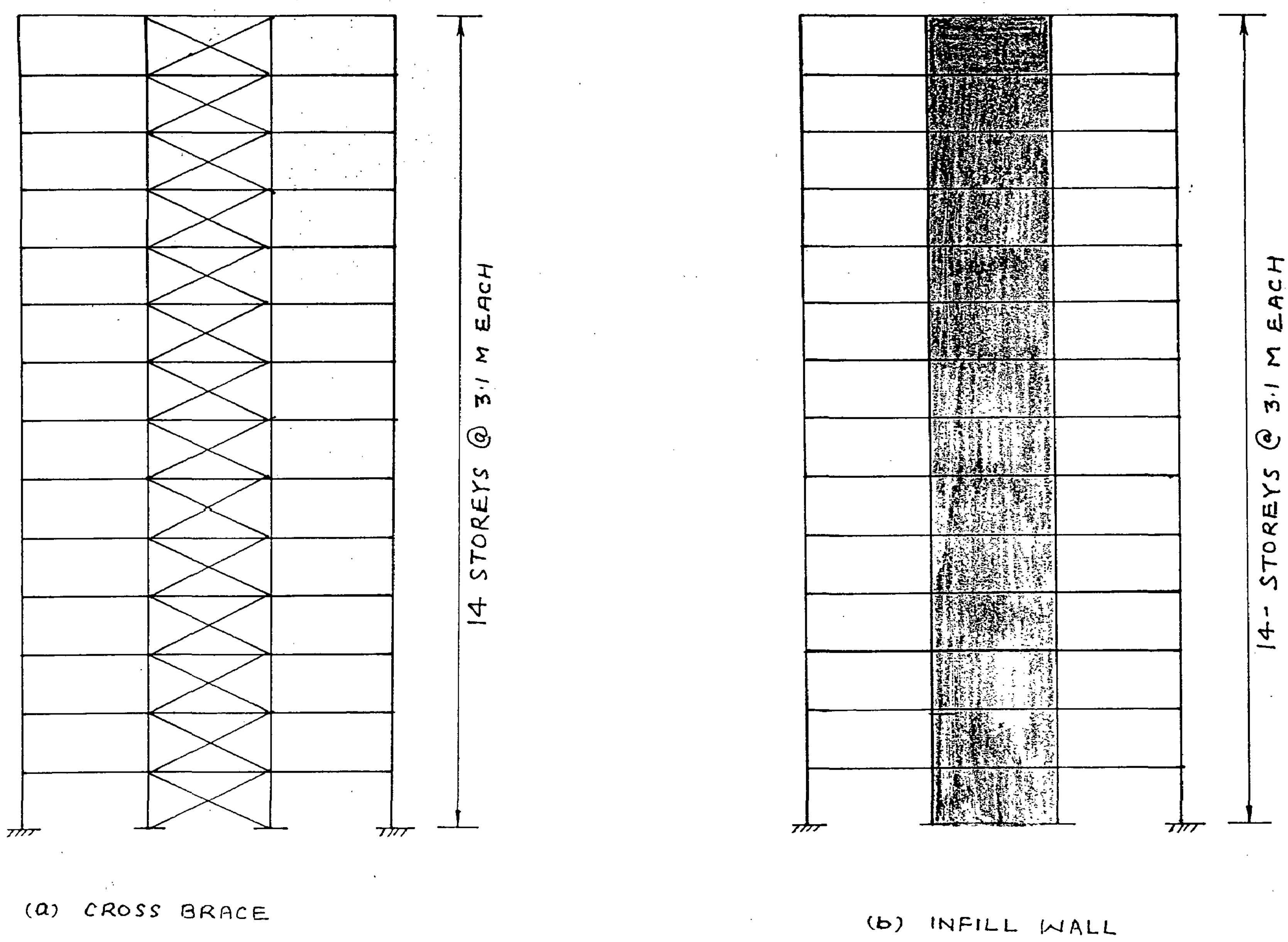


Fig. 4.3 Frame in z-direction with cross brace and infill masonry wall

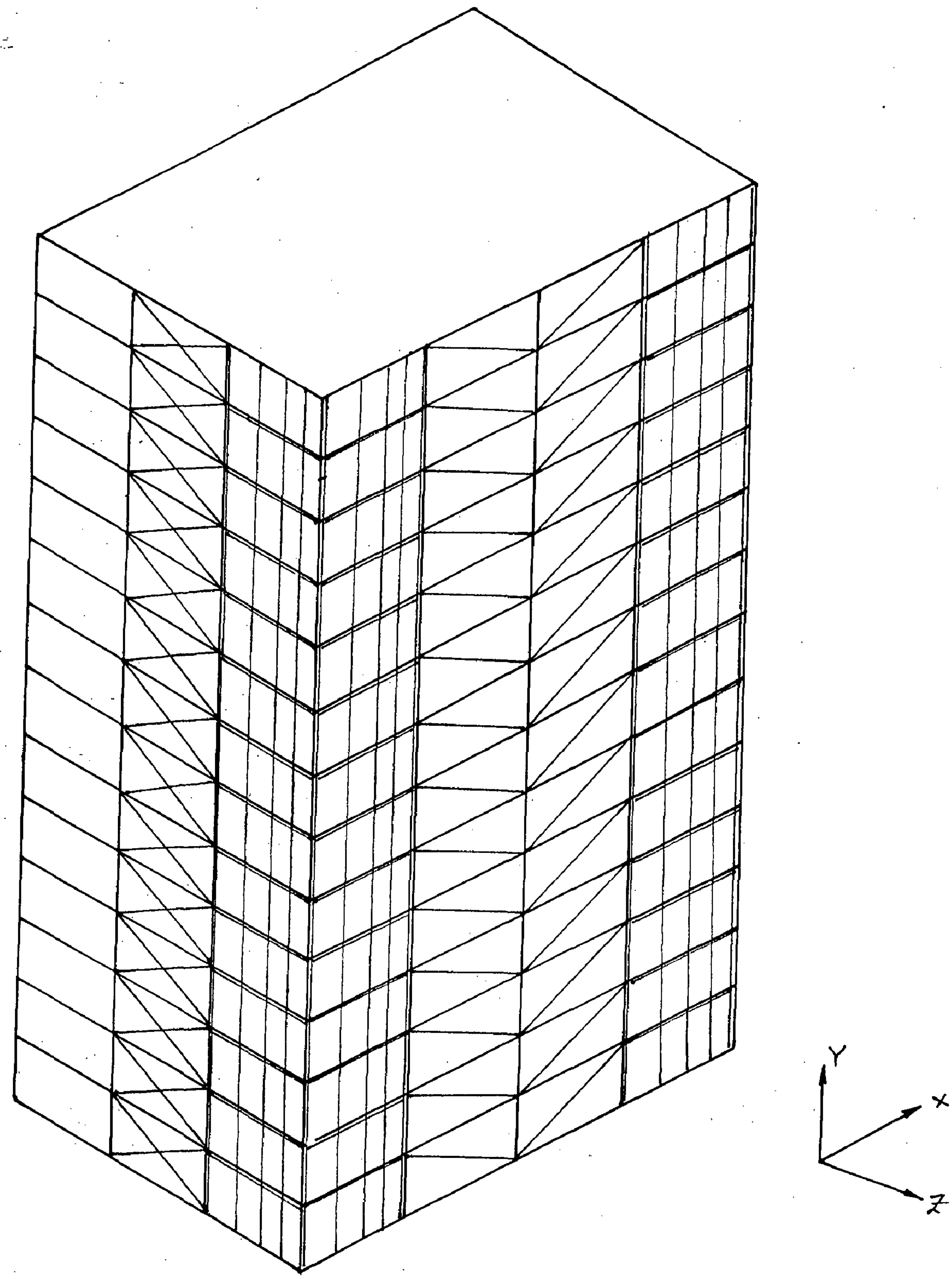


Fig. 4.4 Three dimensional view of existing block B_2 with V-pattern bracing

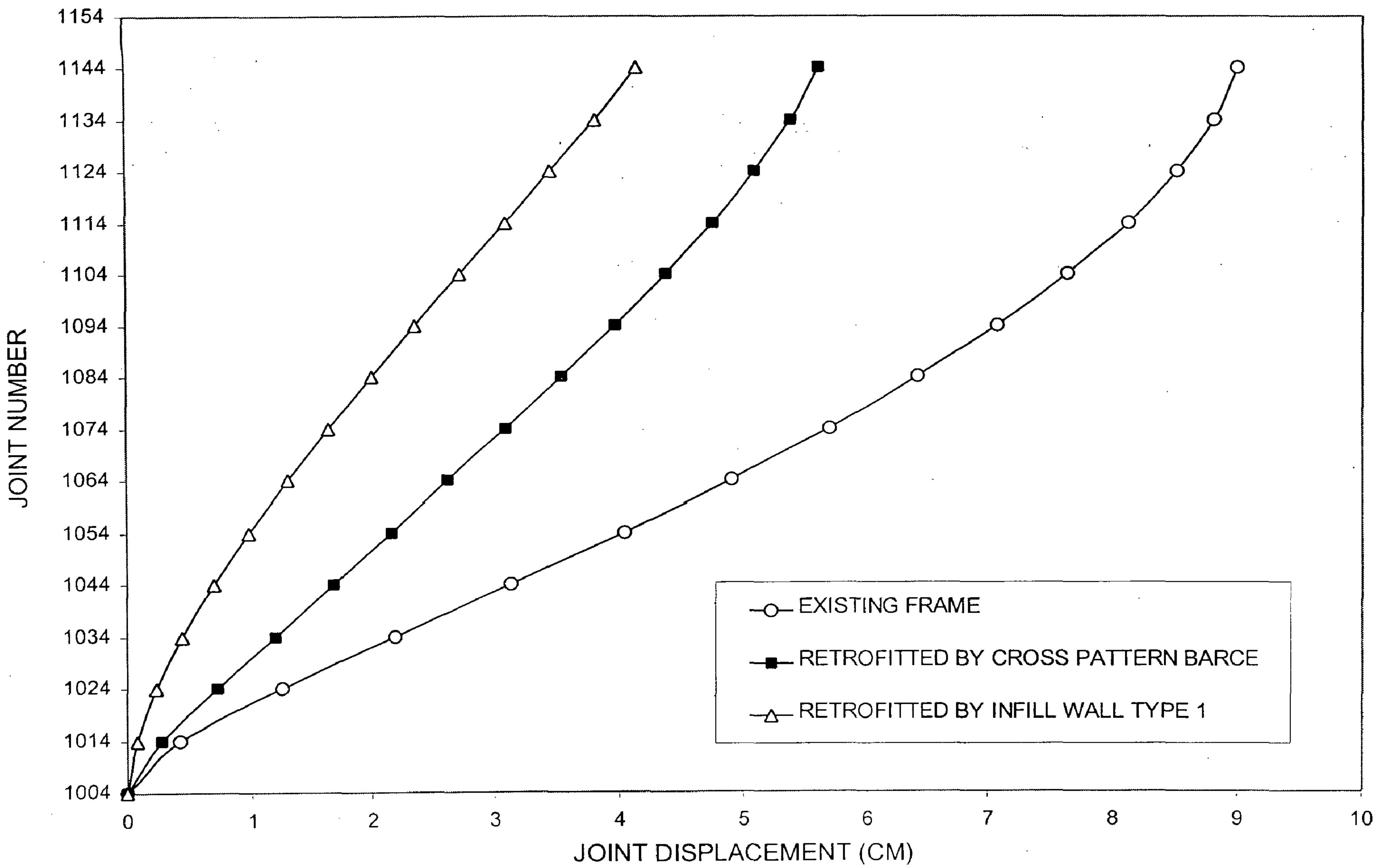


Fig. 4.11 Joint displacement in middle frame at floor level in z-direction

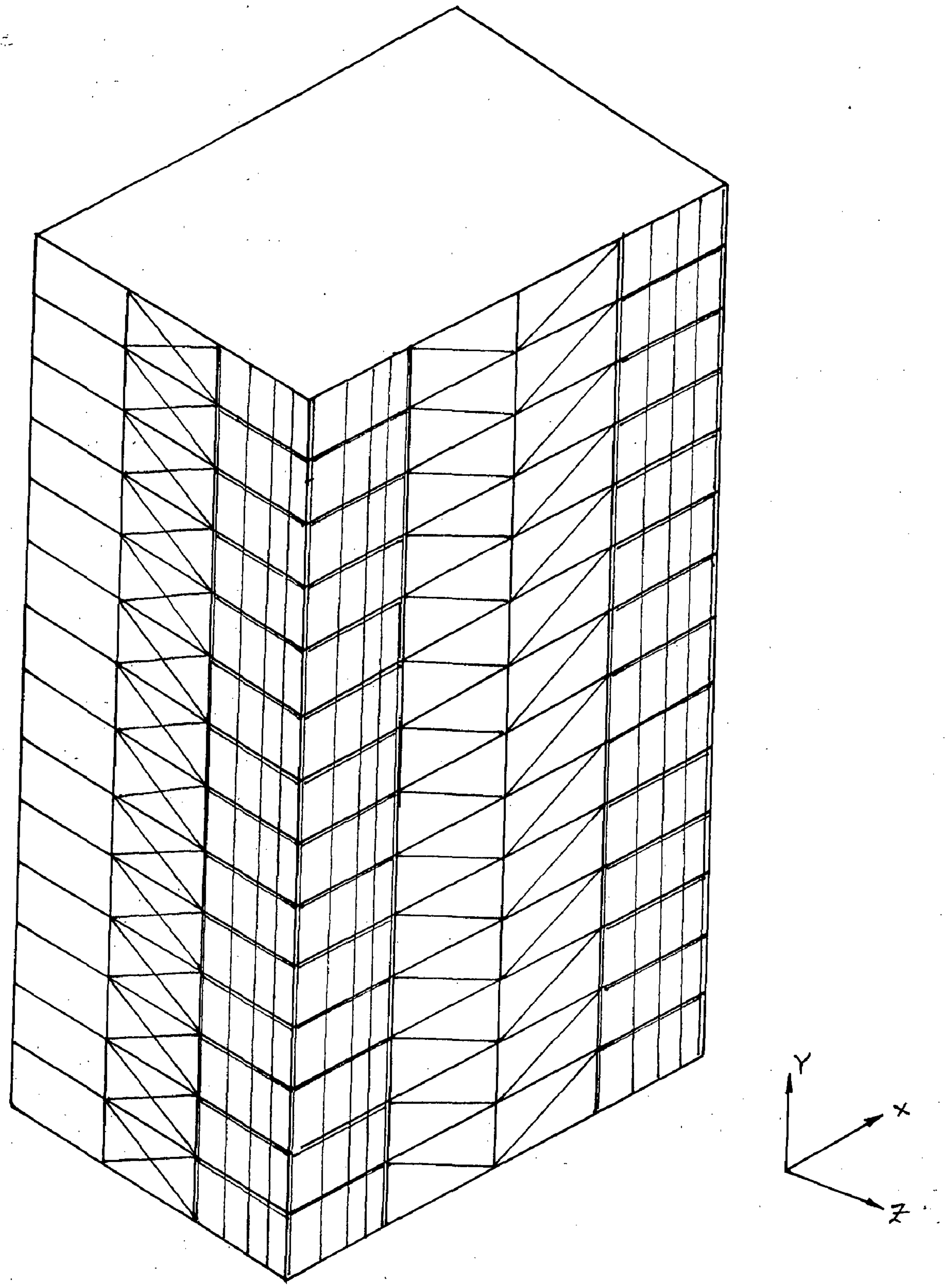


Fig. 4.4 Three dimensional view of existing block B_2 with V-pattern bracing

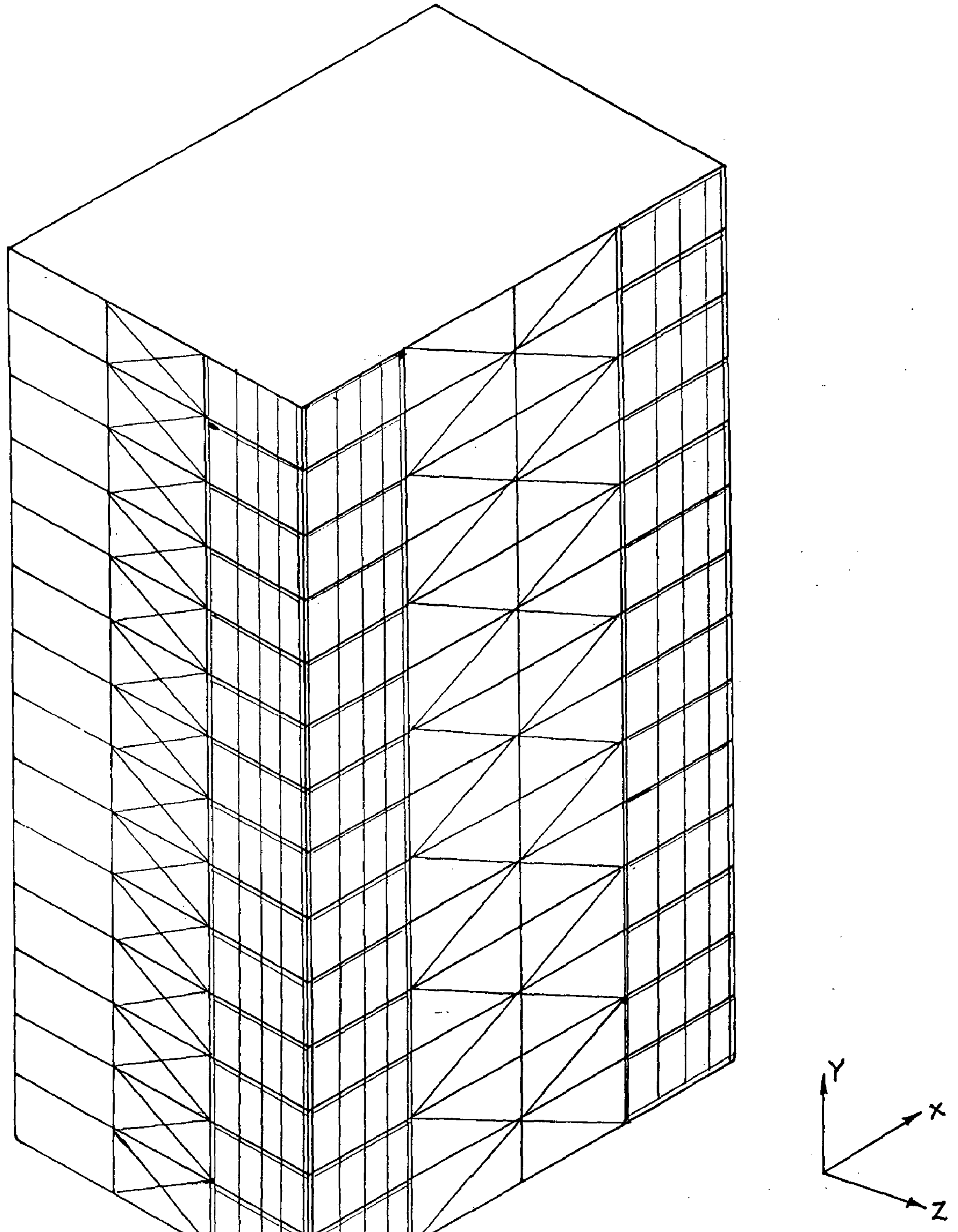


Fig. 4.5 Three dimensional view of existing block B_2 with diamond pattern bracing

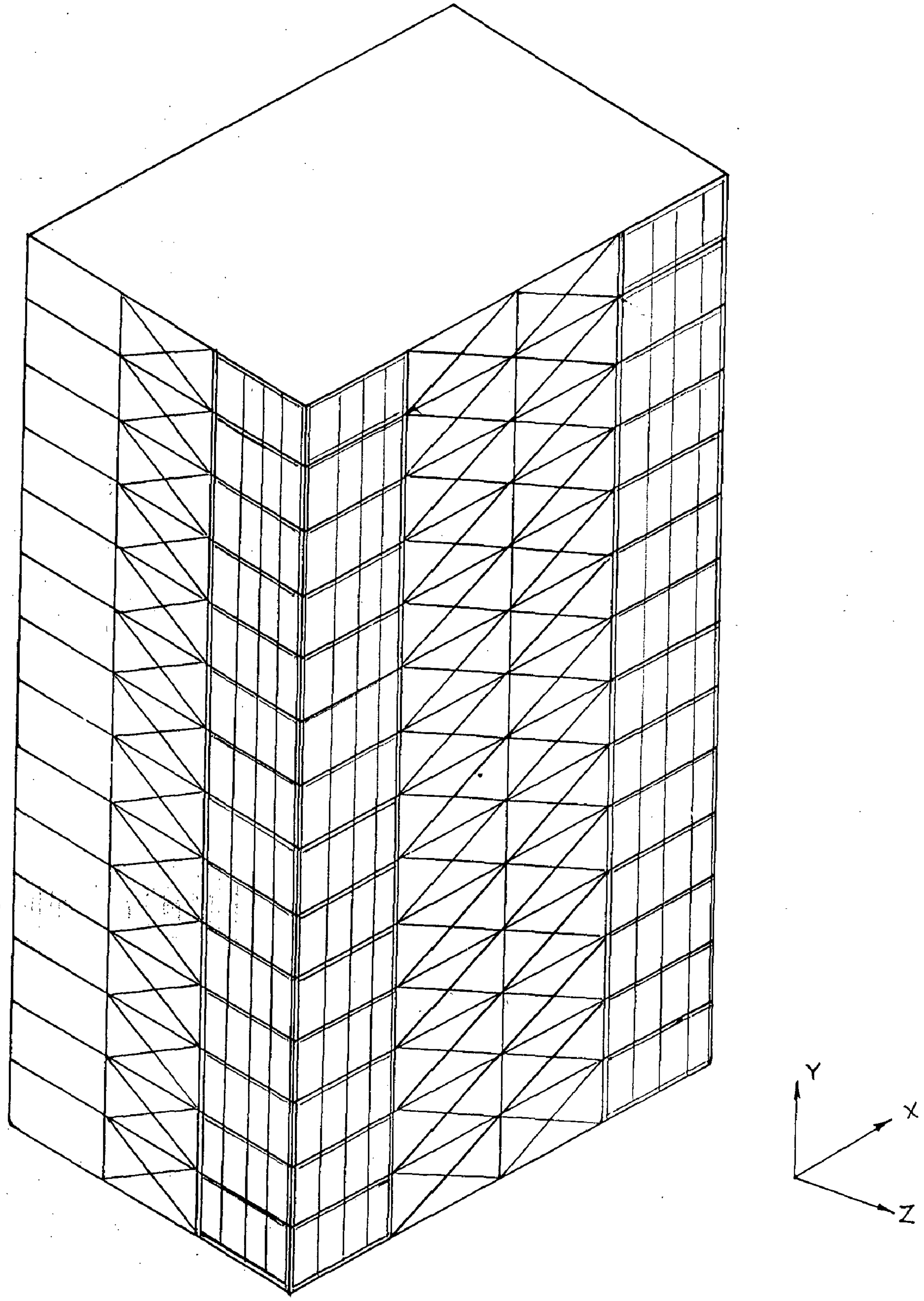


Fig. 4.6 Three dimensional view of existing block B_2 with cross pattern bracing

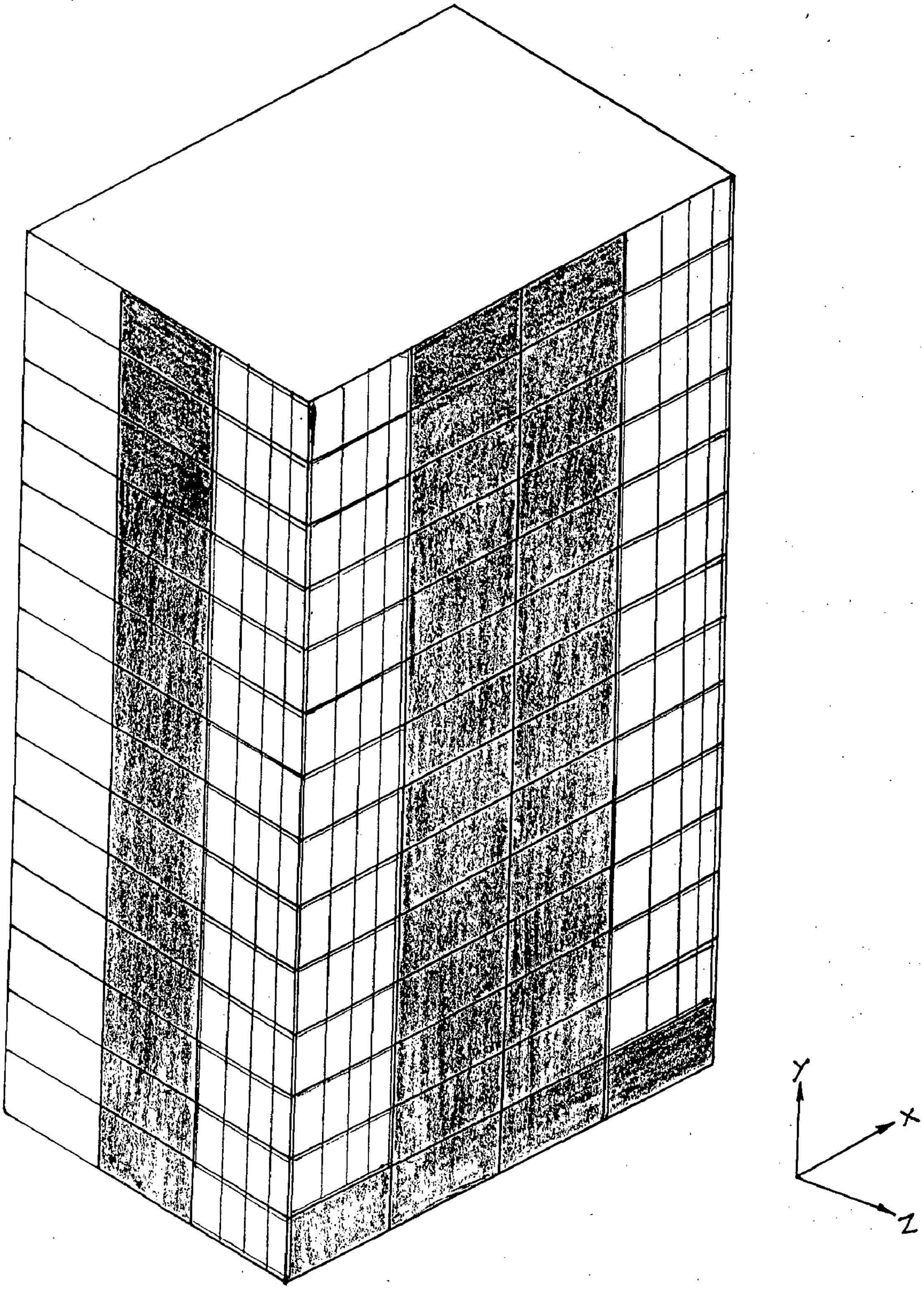


Fig. 4.7 Three dimensional view of existing block B_2 with infill masonry wall type 1

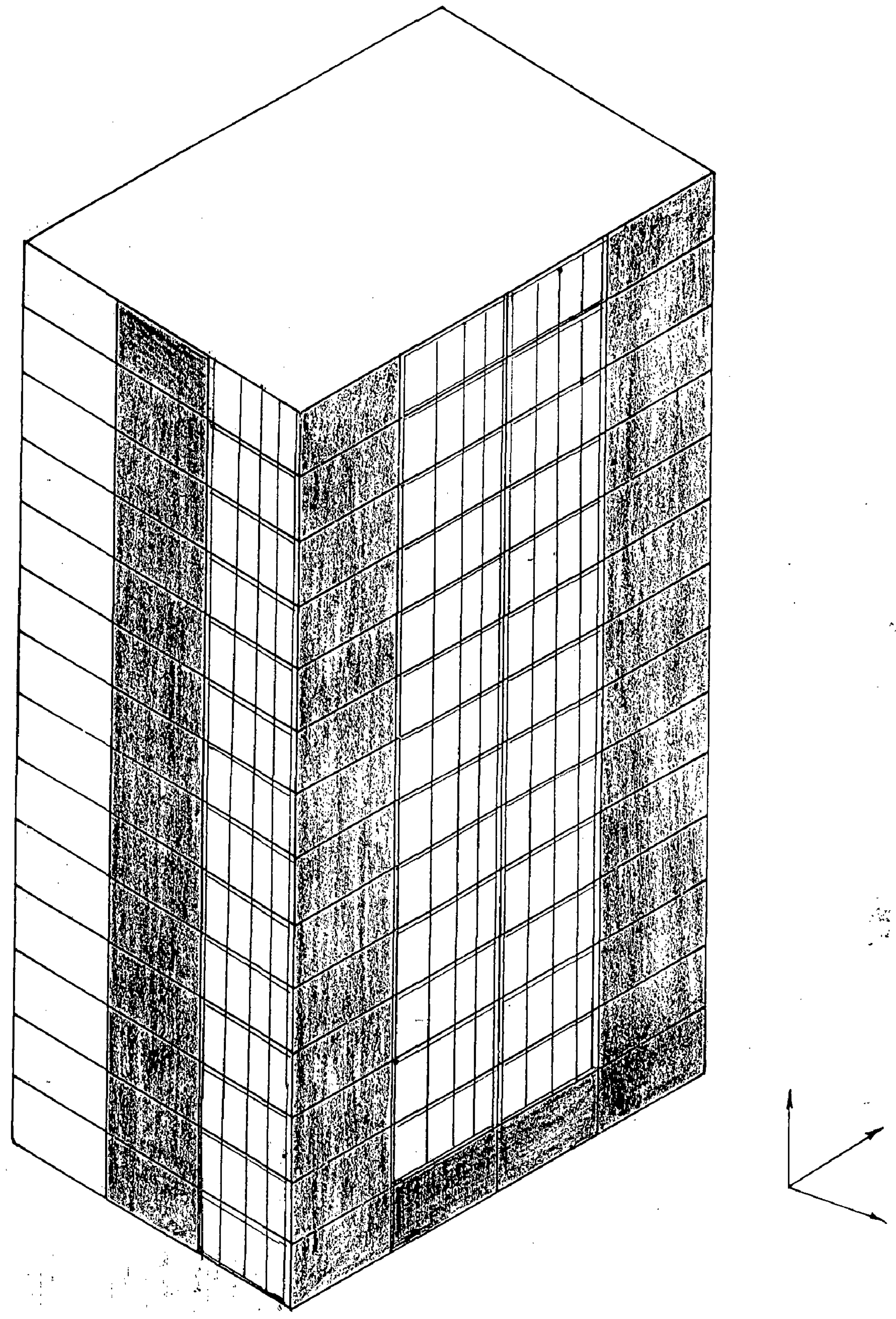


Fig. 4.8 Three dimensional view of existing block B_2 with infill masonry wall type 2

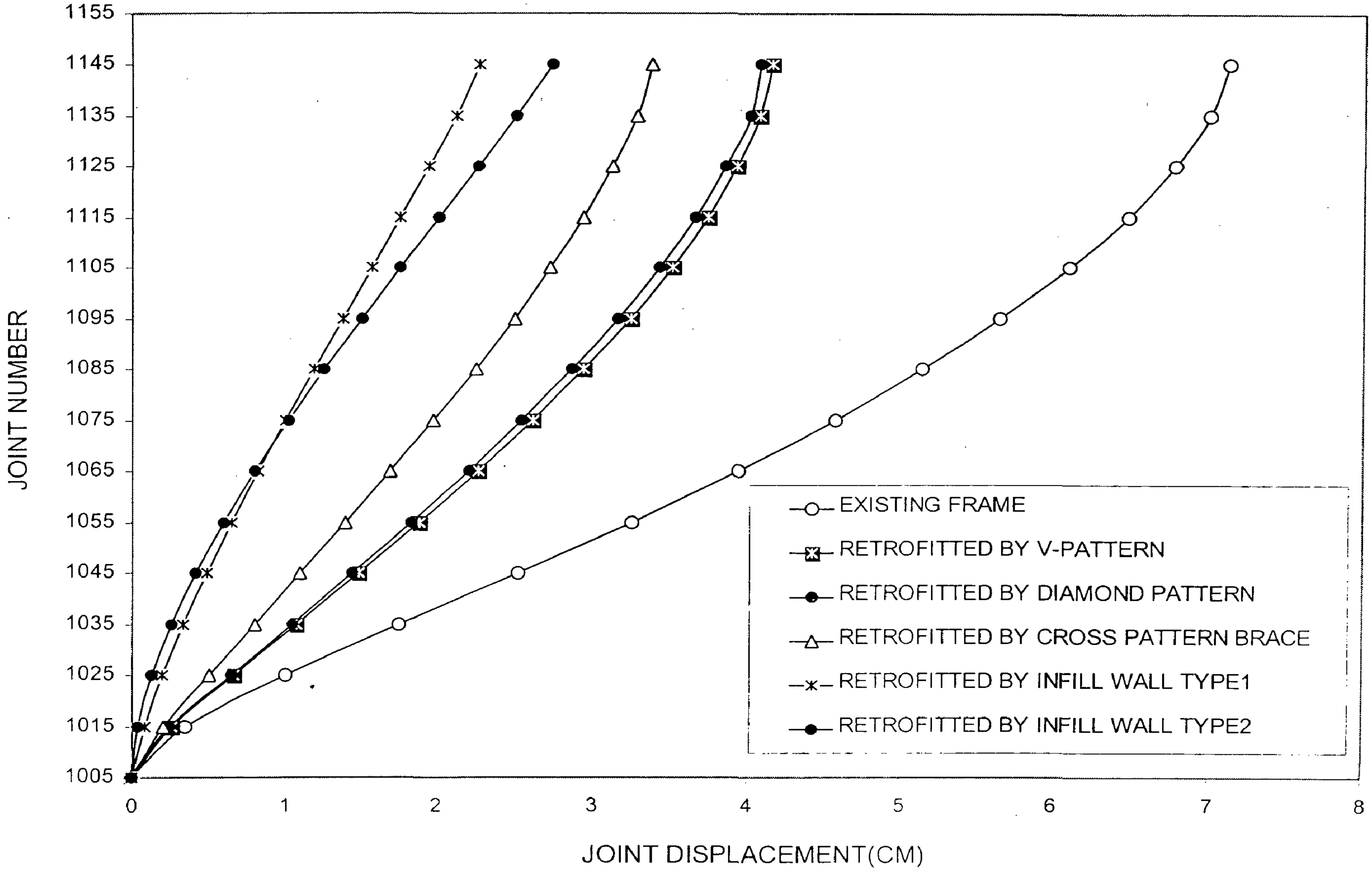


Fig. 4.9 Joint displacement in edge frame at floor levels in x-direction

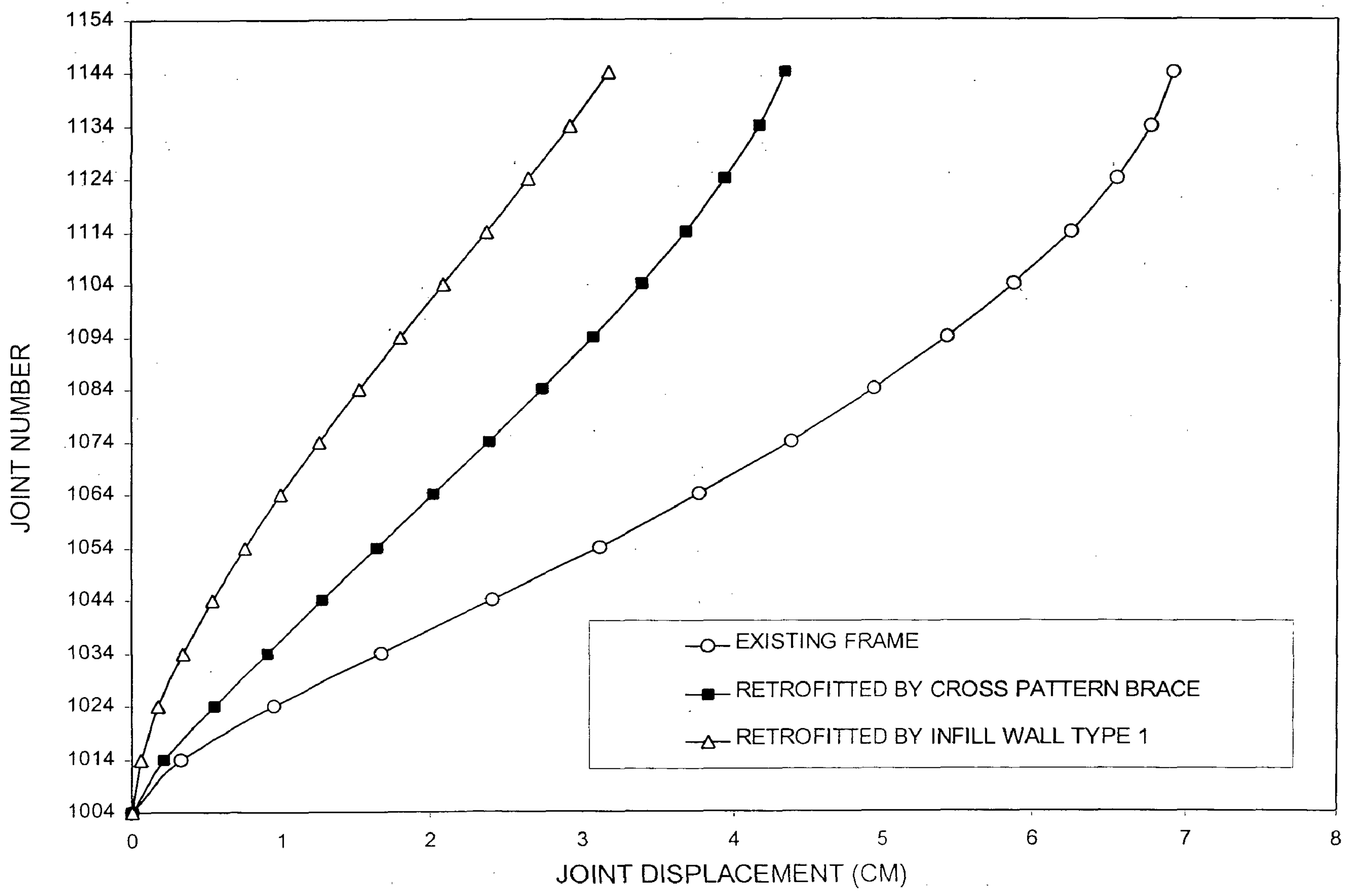


Fig. 4.10 Joint displacement in edge frame at floor levels in z-direction

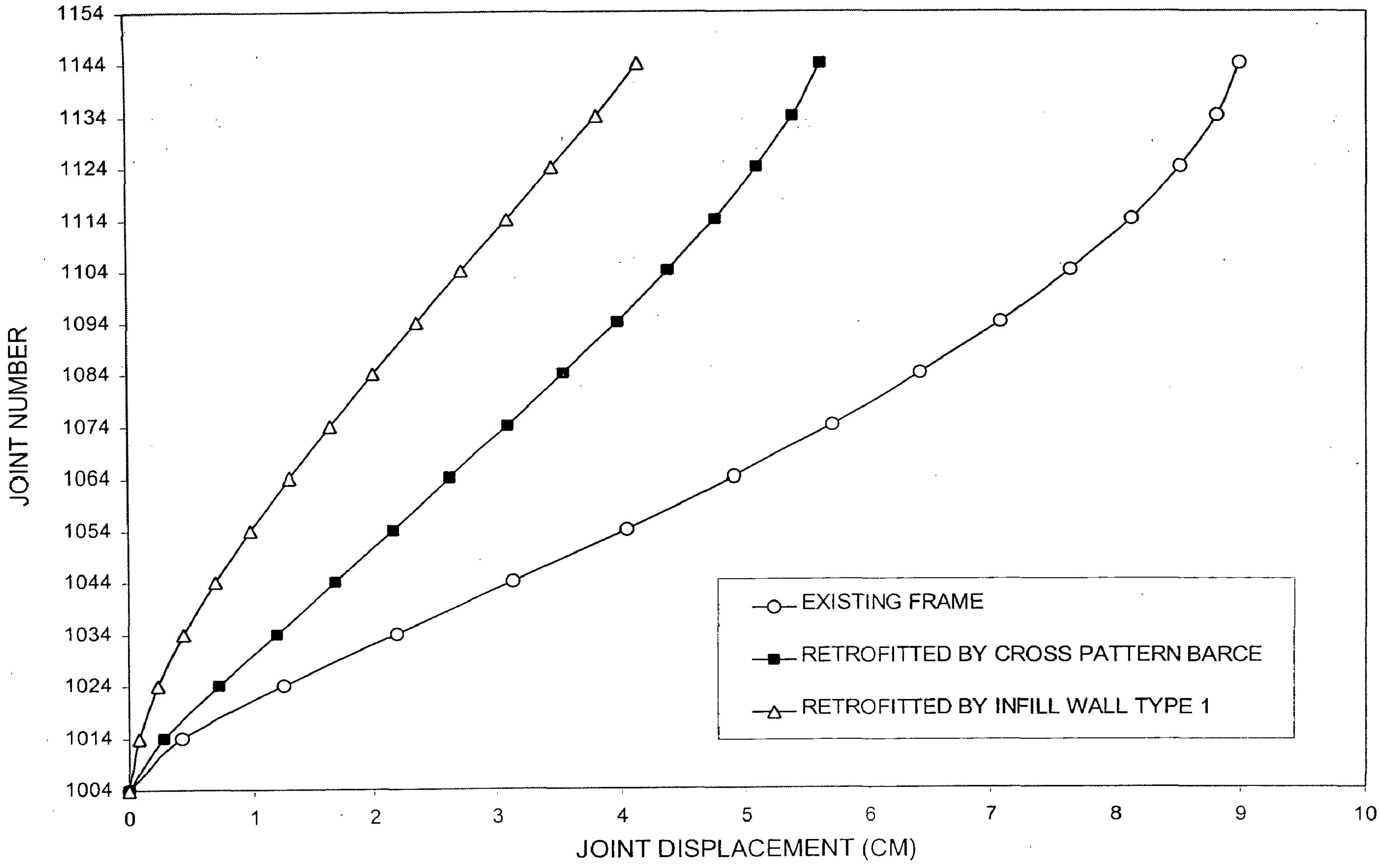


Fig. 4.11 Joint displacement in middle frame at floor level in z-direction

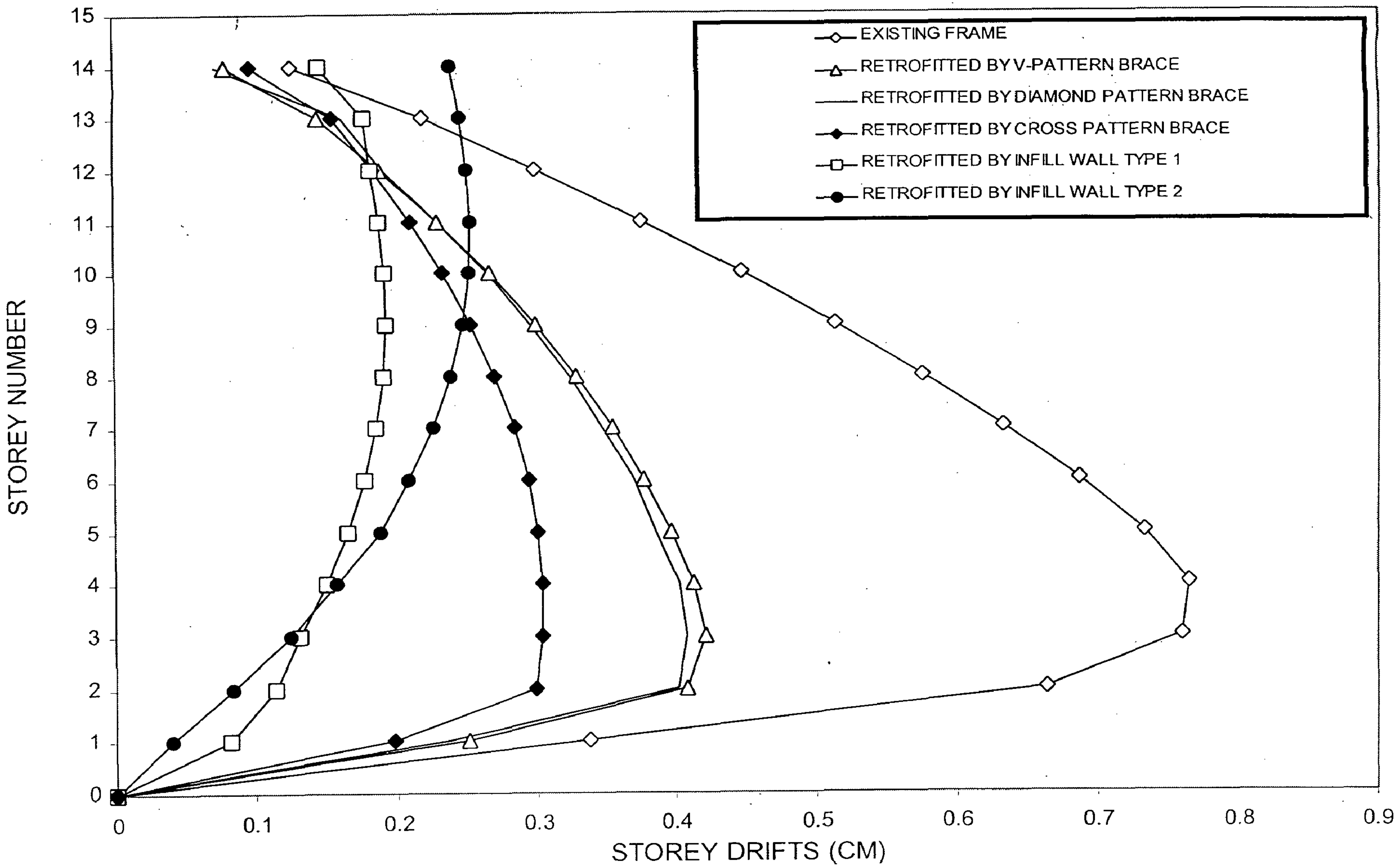


Fig. 4.12 Story drifts in edge frame in x-direction

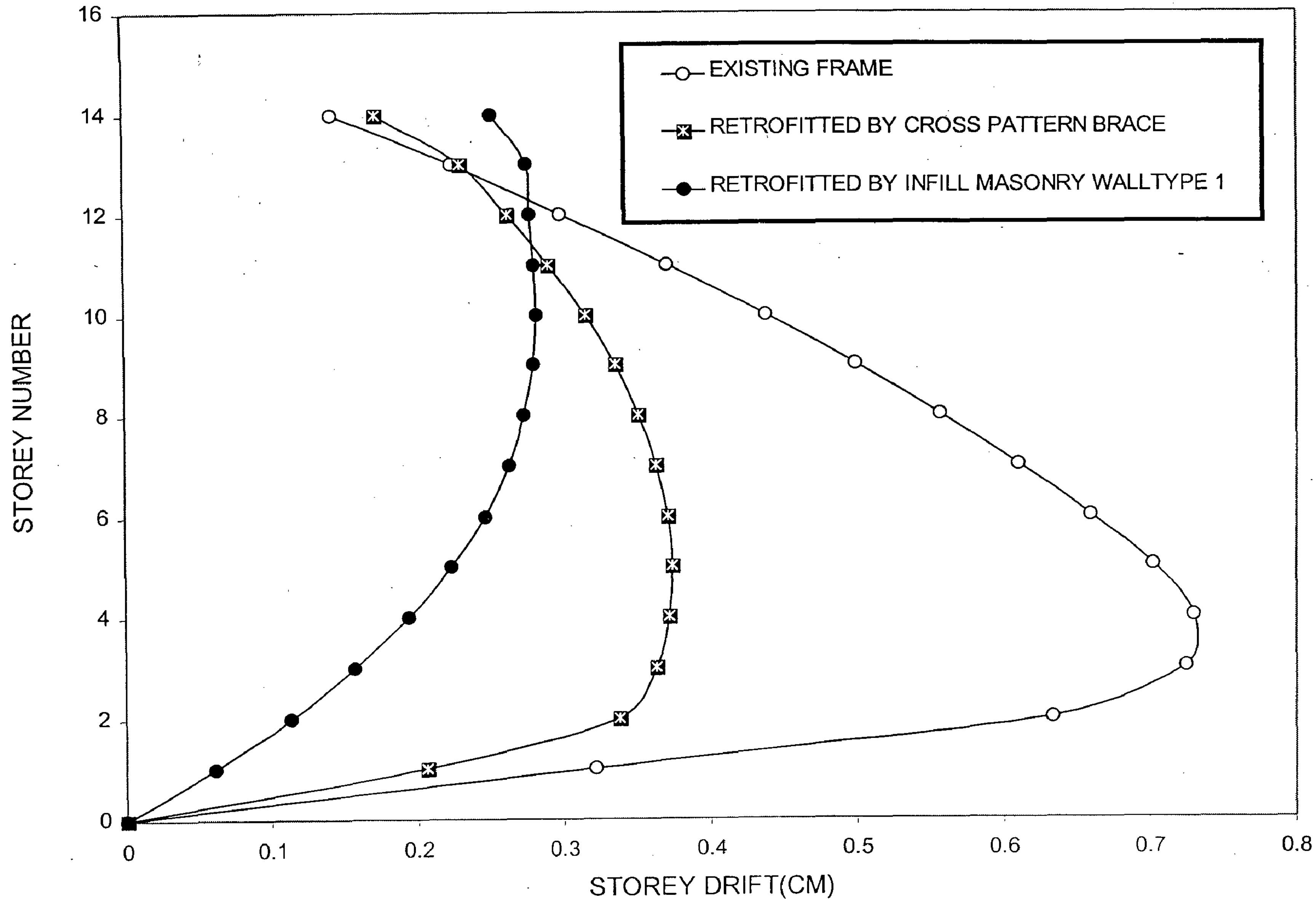


Fig. 4.13 Story drifts in edge frame in z-direction

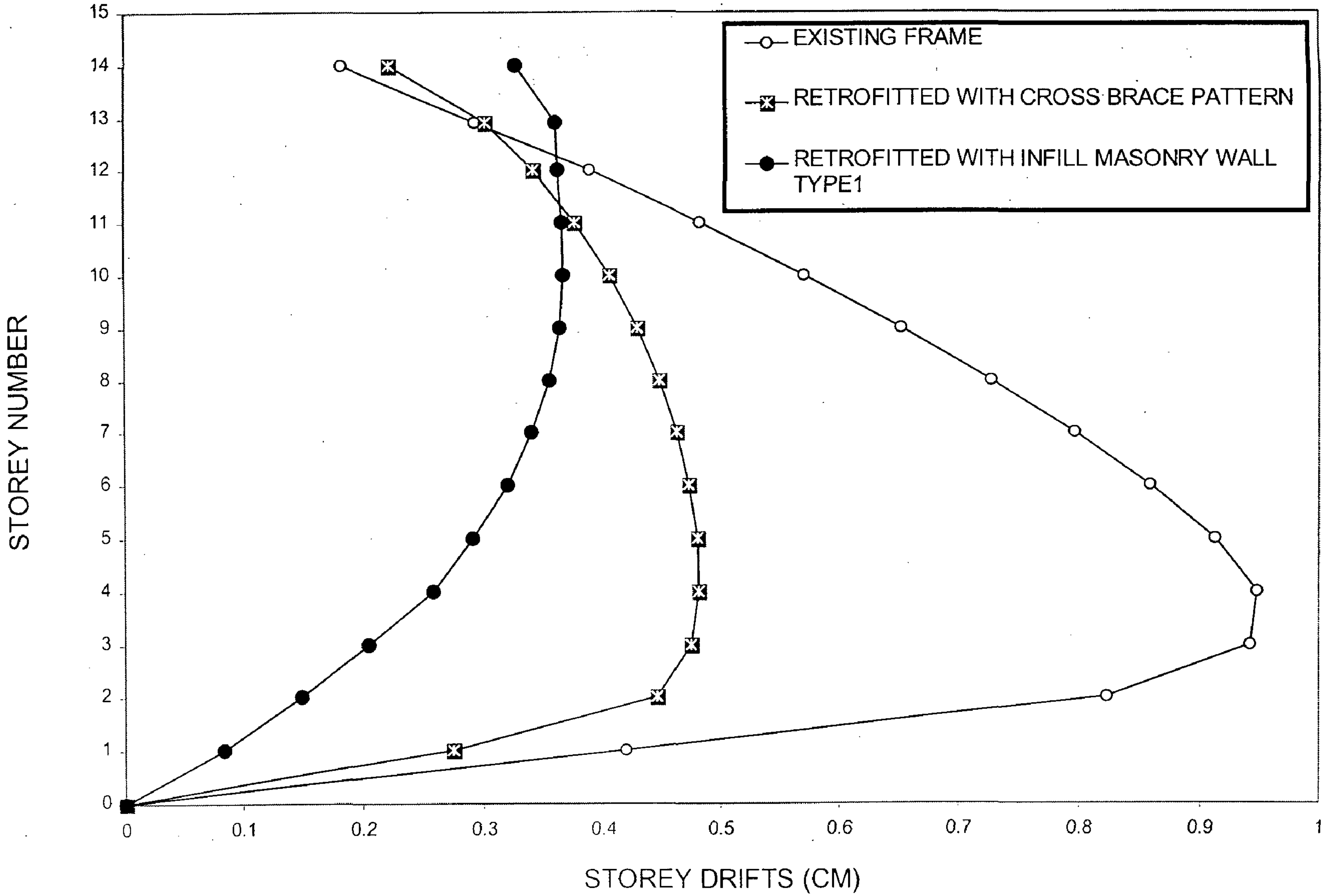


Fig. 4.14 Story drifts in middle frame in z-direction

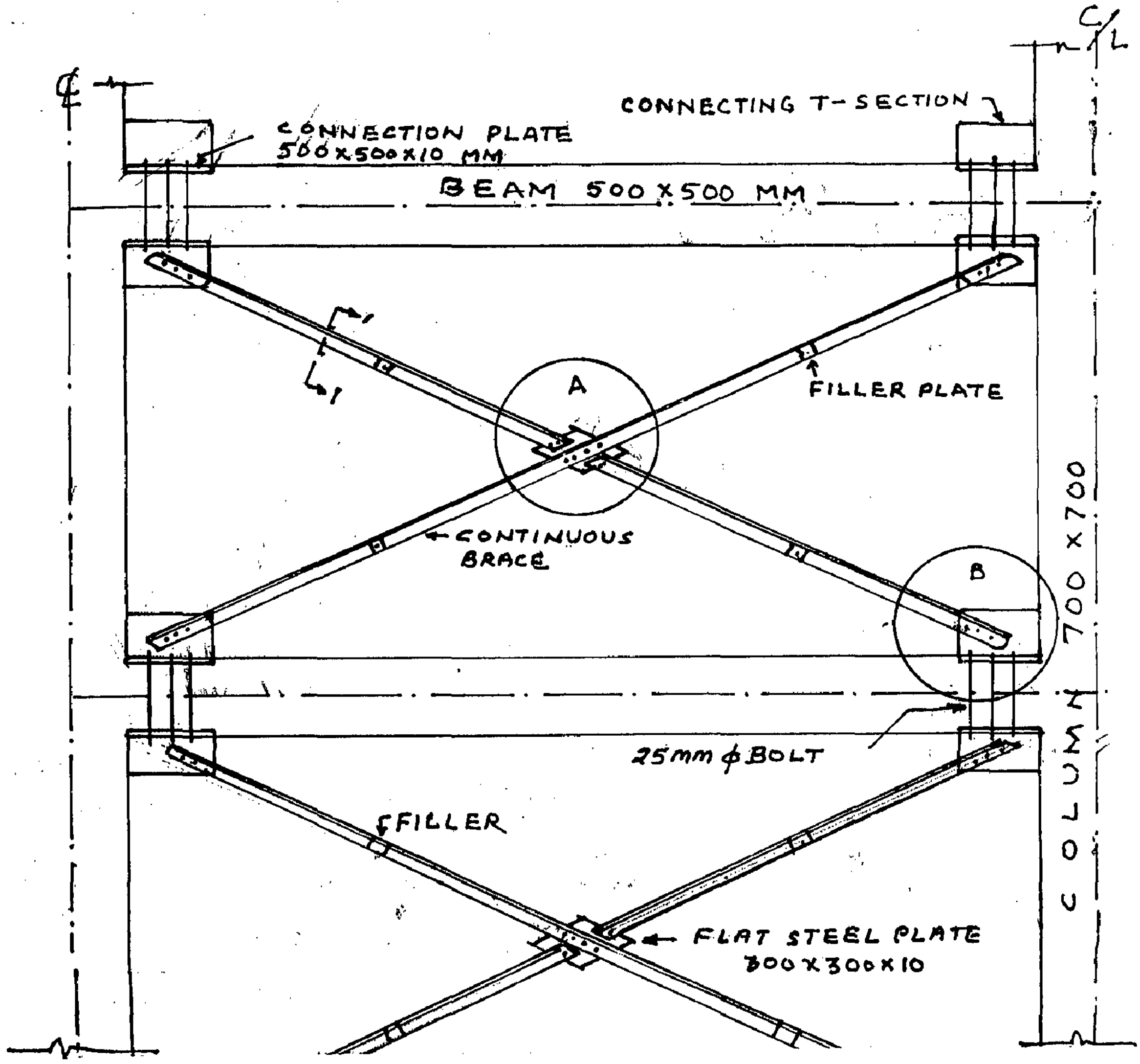
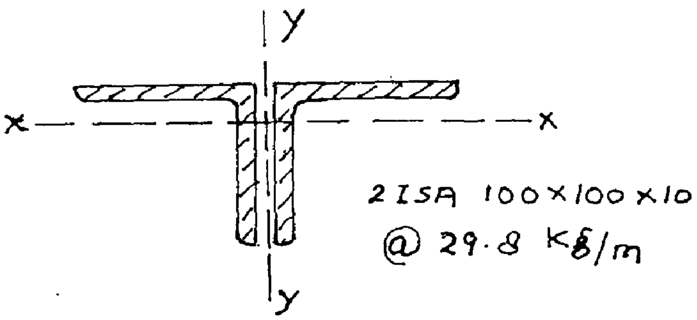
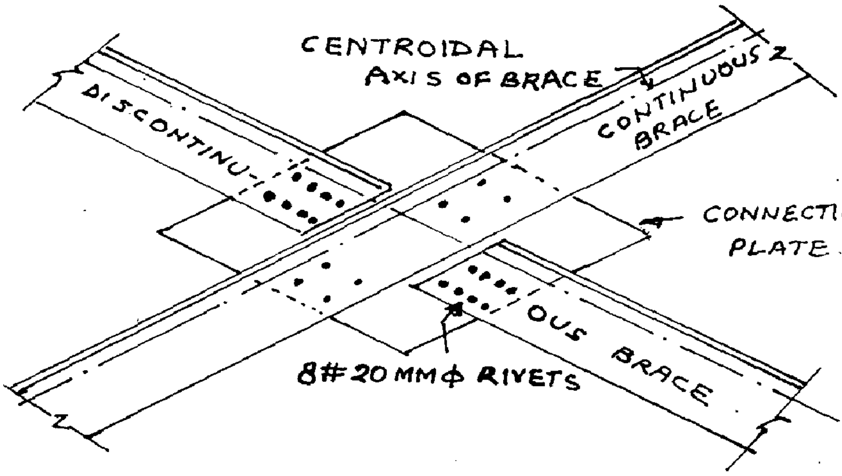


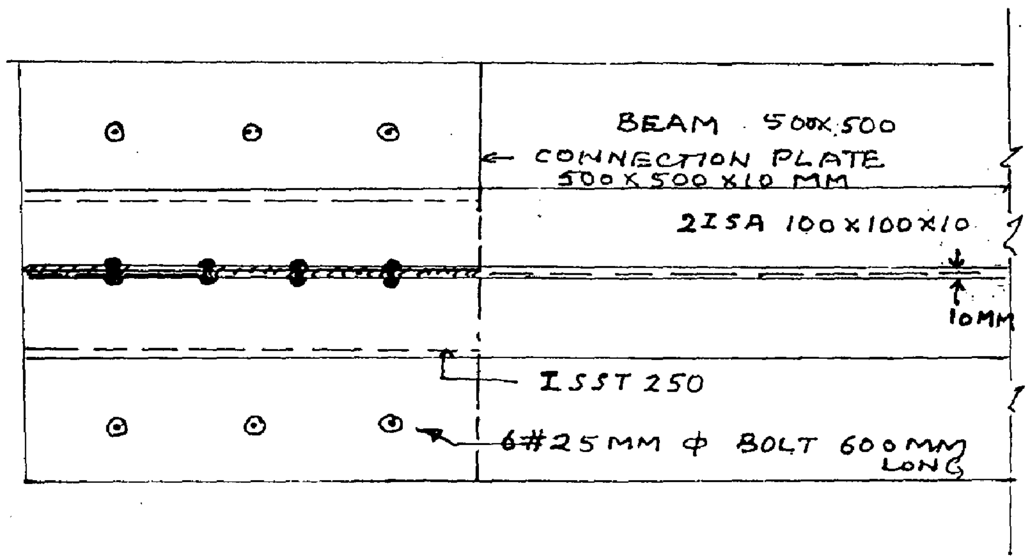
Fig. A.1 (a) Connections for cross brace pattern at first floor



SECTION AT 1-1

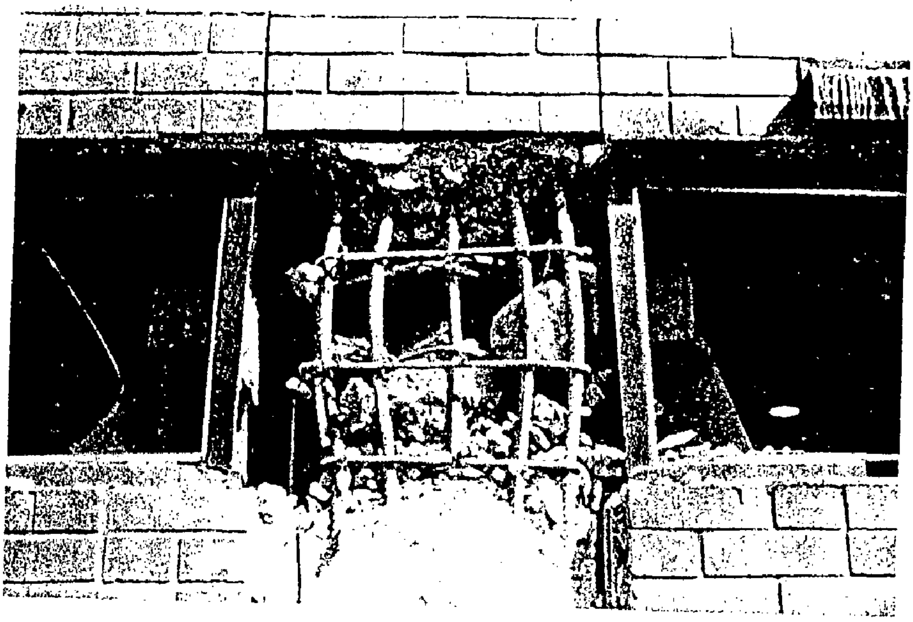


DETAIL AT 'A'

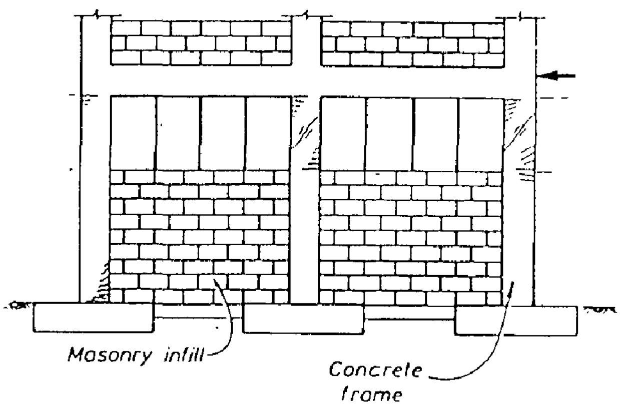


DETAILS AT 'B'

Fig. A.1 (b) Details of show windows as given in Fig. A.1 (a)

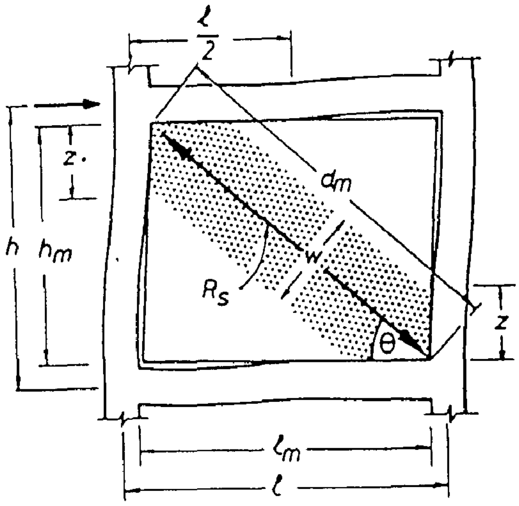


Influence of partial height infill increasing column shear force

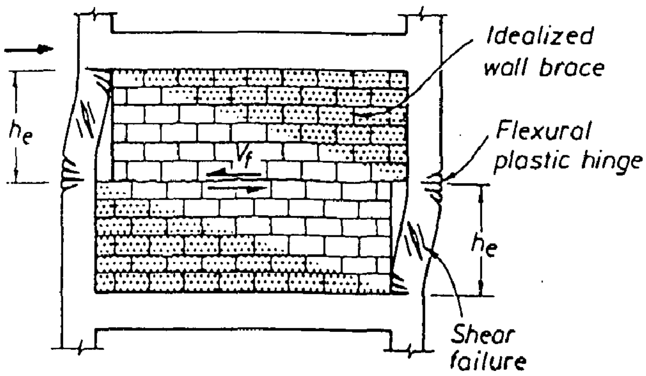


Partial masonry infill in concrete frame

Fig. A.2 (a) Failures due to masonry infill in R.C. frames



Deformation under shear load



sliding shear failure of masonry infill.

Fig. A.2 (b) Frame with infill masonry wall showing notations used in the design

APPENDIX

(A.1) DESIGN OF CROSS BRACE PATTERN

The sample calculation for design and checks for lateral buckling of braces in cross brace pattern. The forces in the diagonal braces at the ground floor for edge frame in x-direction is considered in design as given below:

DATA:

Section of member = 2ISA100×100×10 (back to back spacing 10mm)

Section properties are as follows

Cross-section area A = 38.06 cm^2

Moment of inertia I_{xx} = 354.0 cm^4

Section modulus Z_{xx} = 49.40 cm^3

Radius of gyration r_{xx} = 3.05 cm

Radius of gyration r_{yy} = 4.52 cm

C. G. distance C_{xx} = 2.84 cm

Weight per unit length w = 29.8 kg/m

Effective length l_{eff} = 0.85 l

Total diagonal length = 6.356 m

Length of brace L = 6.0 m

Due to connection at intersection, considering the length of brace

Slenderness ratio $\lambda = L/r_{min}$

$\therefore \quad = 6000/30.5$

$= 196.72$

The compression brace is safe for buckling, because $R_S < \pi^2/3$.

The actual value of effective length factor,

$$k = \frac{1}{1+3\left(\frac{K_T}{K_0}\right)}$$
$$k = \frac{1}{1+1.159}$$
$$k = 0.4637$$

Elastic buckling stress,

$$f_{cr} = \frac{\pi^2 E}{\left(\frac{kL}{r_{min}}\right)^2}$$
$$f_{cr} = \frac{\pi^2 \times 2\times 10^5}{\left(\frac{0.463\times 6000}{3.05\times 10}\right)^2}$$
$$f_{cr} = 237.76 N/mm^2$$

Stress ratio, $\quad \frac{f_{at}}{f_{cr}} = \frac{125.06}{237.76}$

$= 0.526 \quad$ (i.e. less than 0.6)

Hence the braces are safe for buckling for the adopted size of the section.

a) **DESIGN OF COMPRESSION BRACE**

Section properties for 2ISA100×100×10 (back to back at spacing 10 mm)

Radii of gyration $\quad r_{xx} = 3.05$ cm

$r_{yy} = 4.52$ cm

Effective length factor $k = 0.85$

Slenderness ratio $\lambda = L/r_{xx}$

$= 0.85 \times 3000/30.5$

$= 83.6$

Yielding stress for steel $f_y = 250\ N/mm^2$

Allowable stress from table 5.1(IS:800-1984)

$$\sigma_{ac} = 97.04 N/mm^2$$

$\therefore$ Load carrying capacity of compression diagonal

$= 3806 \times 97.04$

$= 369.33$ KN

Therefore provided brace member as (2ISA100×100×10 back to back at spacing 10 mm) is O.K.

b) DESIGN OF CONNECTIONS

(i) For compression

Using 20mm diameter power driven field rivets

Gross diameter of rivet $= 20+1.5$

$= 21.5$ mm

Strength of rivet in double shear $= 2 \times (\pi/4) \times f_s \times d^2$

$= 2 \times (\pi/4) \times 21.5^2 \times 100/1000$

$= 72.6$ KN

Strength of rivet in bearing $= f_b \times d \times t$

$= 21.5 \times 10 \times 300/1000$

$= 64.5$ KN

Rivet value(minimum of above strength) $R_v = 64.5$ KN

$\therefore$ Number of rivets required $n = P/R_v$

$= 294.24/64.5$

$= 6$

(ii) For Tension

Using 20 mm diameter power driven field rivets

Allowable stress in axial tension $= 0.6 \times f_y$

$= 150$ N/mm^2

Net area required for tension brace, $A_{net} = 294240/150$

$= 1961.60$ mm^2

Provided gross area $A_{gross} = 3806$ mm^2

Provided net area $= 3806 - 2 \times 21.5 \times 10$

$= 3376$ mm^2

Therefore maximum tensile load capacity $= 150 \times 3376$

$= 506.40$ KN > 294.24 KN (O.K.)

$\therefore$ Number of rivets required for tension $= 506.40/64.5$

$= 8$

Hence providing 8 rivets of 20-mm diameter in two rows (4 in each row).

APPENDIX

(A.1) DESIGN OF CROSS BRACE PATTERN

The sample calculation for design and checks for lateral buckling of braces in cross brace pattern. The forces in the diagonal braces at the ground floor for edge frame in x-direction is considered in design as given below:

DATA:

Section of member = 2ISA100×100×10 (back to back spacing 10mm)

Section properties are as follows

Cross-section area A = 38.06 cm^2

Moment of inertia I_{xx} = 354.0 cm^4

Section modulus Z_{xx} = 49.40 cm^3

Radius of gyration r_{xx} = 3.05 cm

Radius of gyration r_{yy} = 4.52 cm

C. G. distance C_{xx} = 2.84 cm

Weight per unit length w = 29.8 kg/m

Effective length l_{eff} = 0.85 l

Total diagonal length = 6.356 m

Length of brace L = 6.0 m

Due to connection at intersection, considering the length of brace

In compression $\quad L = 6.0/2$

$= 3.0$ m

In tension $\quad = 6.0$ m

Maximum axial force in member as per analysis

$P = \pm 294.24$ KN

a) **Checking for lateral buckling**

Lateral stiffness of tension member, $K_L = \dfrac{4P}{L}\left[\dfrac{1}{1-\left(\dfrac{2}{aL}\right).\tanh\left(\dfrac{aL}{2}\right)}\right]$

Where, $a = \sqrt{\dfrac{P}{EI}}$ and $\tanh\left(\dfrac{aL}{2}\right) = \dfrac{e^{\frac{aL}{2}} - e^{\frac{-aL}{2}}}{e^{\frac{aL}{2}} + e^{\frac{-aL}{2}}}$

$$a = \sqrt{\frac{294.24\times10^3}{2\times10^5\times354\times10^4}}$$

$$a = 819.95\times10^{-6}\,mm^{-1}$$

$$\frac{aL}{2} = 819.95\times10^{-6}\times6000/2 = 1.934$$

$\therefore$ $\tanh\left(\dfrac{aL}{2}\right) = 0.959$

$\therefore$ $K_L = 389.12KN/M$

Bending stiffness of tension member, $K_0 = \frac{48EI}{L^3}$

$$\Rightarrow \quad K_0 = \frac{48 \times 2 \times 10^5 \times 354 \times 10^4}{(6000)^3}$$

$$K_0 = 157.33 KN/M$$

Therefore, lateral stiffness ratio $\quad R_S = \frac{K_L}{K_0}$

$$R_S = 2.473$$

<u>For buckling of braced compression diagonal</u>

According to Mutton and Trahair the following relation gives lateral stiffness ratio for compression diagonal brace assuming that flexural rigidity is same for compression and tension diagonals:

Lateral stiffness ratio R_T,
$$\frac{K_T}{K_0} = \frac{\frac{1}{3}\left(\frac{\pi}{2k}\right)^3 \cot\left(\frac{\pi}{2k}\right)}{\frac{\pi}{2k}\cot\left(\frac{\pi}{2k}\right) - 1}$$

Where, k = effective length factor

k = 0.85

On substituting values in above equation, we get

$$\frac{K_T}{K_0} = 1.159$$

Tensile stress f_{at} = P/A

Provided gross area A = 3806 mm^2 (i.e. cross-section area of 2ISA100×100×10 placing back to back at an spacing of 10 mm.

∴ Tensile stress
$$f_{at} = \frac{476 \times 10^3}{3806}$$

$$f_{at} = 125.06 N/mm^2$$

Slenderness ratio $\lambda = L/r_{min}$

$\therefore \quad = 6000/30.5$

$= 196.72$

The compression brace is safe for buckling, because $R_S < \pi^2/3$.

The actual value of effective length factor,

$$k = \frac{1}{1+3\left(\frac{K_T}{K_0}\right)}$$
$$k = \frac{1}{1+1.159}$$
$$k = 0.4637$$

Elastic buckling stress,

$$f_{cr} = \frac{\pi^2 E}{\left(\frac{kL}{r_{min}}\right)^2}$$
$$f_{cr} = \frac{\pi^2 \times 2 \times 10^5}{\left(\frac{0.463 \times 6000}{3.05 \times 10}\right)^2}$$
$$f_{cr} = 237.76 N/mm^2$$

Stress ratio, $$\frac{f_{at}}{f_{cr}} = \frac{125.06}{237.76}$$

$= 0.526$ (i.e. less than 0.6)

Hence the braces are safe for buckling for the adopted size of the section.

a) **DESIGN OF COMPRESSION BRACE**

Section properties for 2ISA100×100×10 (back to back at spacing 10 mm)

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$r_{yy} = 4.52$ cm

Effective length factor $k = 0.85$

Slenderness ratio $\lambda = L/r_{xx}$

$= 0.85 \times 3000/30.5$

$= 83.6$

Yielding stress for steel $f_y = 250 \text{ N/mm}^2$

Allowable stress from table 5.1(IS:800-1984)

$$\sigma_{ac} = 97.04 N/mm^2$$

$\therefore$ Load carrying capacity of compression diagonal

$= 3806 \times 97.04$

$= 369.33$ KN

Therefore provided brace member as (2ISA100×100×10 back to back at spacing 10 mm) is O.K.

b) DESIGN OF CONNECTIONS

(i) For compression

Using 20mm diameter power driven field rivets

Gross diameter of rivet $= 20+1.5$

$= 21.5$ mm

Strength of rivet in double shear $= 2 \times (\pi/4) \times f_s \times d^2$

$= 2 \times (\pi/4) \times 21.5^2 \times 100/1000$

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Strength of rivet in bearing $= f_b \times d \times t$

$= 21.5 \times 10 \times 300/1000$

$= 64.5$ KN

Rivet value(minimum of above strength) $R_v = 64.5$ KN

$\therefore$ Number of rivets required $n = P/R_v$

$= 294.24/64.5$

$= 6$

(ii) For Tension

Using 20 mm diameter power driven field rivets

Allowable stress in axial tension $= 0.6 \times f_y$

$= 150$ N/mm^2

Net area required for tension brace, $A_{net} = 294240/150$

$= 1961.60$ mm^2

Provided gross area $A_{gross} = 3806$ mm^2

Provided net area $= 3806 - 2 \times 21.5 \times 10$

$= 3376$ mm^2

Therefore maximum tensile load capacity $= 150 \times 3376$

$= 506.40$ KN > 294.24 KN (O.K.)

$\therefore$ Number of rivets required for tension $= 506.40/64.5$

$= 8$

Hence providing 8 rivets of 20-mm diameter in two rows (4 in each row).

c) DESIGN OF THREADED ROD BOLT

Providing 25-mm diameter and 600 mm long steel rods having threads (therefore named as threaded rod bolt) at the ends to connect the top and bottom connecting plates.

Assuming that these bolts will subject to single shear failure due to horizontal component of axial tension or compression in braces.

Angle of diagonal brace to horizontal $\theta = \tan^{-1}(2.6/5.8)$

$= 24.145^{0.}$

Shear force in bolt $= P\cos\theta$

$= 294.24 \cos(24.145)$

$= 268.49$ KN

Strength of bolt in single shear $= 100\times25^2\times\pi/4$

$= 49.0$ KN

$\therefore$ Number of bolts required $= 268.49/49.0$

$\cong 6$

Providing 6 bolts of 25 mm diameter 3 on each side.

Fig. A.1 (a) shows the cross brace pattern in frames with adopted connections as per above designs and Fig. A.1 (b) shows the further details referred to Fig. A.1 (a).

(A.2) DESIGN OF INFILL MASONRY WALL

The sample calculations for design of masonry wall/panel at ground floor level in x-direction are given below:

DATA :

Strength of masonry $f'_m = 24$ MPa

Elasticity modulus $E_m = 600 \times f'_m$

$= 14.4 \times 10^3$ MPa

Shear strength $\tau_0 = 0.04 \times f'_m$

$= 0.96$ MPa

Assuming that the panel will fail in strut action which is the critical case for panel design.

Clear height of panel $= 3.1 - 0.5$

$= 2.6$ m

Clear length of panel $l_m = 6.5 - 0.7$

$= 5.8$ m

Diagonal length of strut $d_m = 6.356$ m

Width of diagonal strut $W = 0.25 \times d_m$

$= 1.6$ m

$\theta = \tan^{-1}(2.6/5.8)$

$= 24.145^0$

Assuming that the flexure hinging will be at mid height of the column.

Therefore $h_e = h/2$

$= 1.55$ m

Maximum shear force for infill panel will be maximum of the following:

(i) $V_f = \tau_0 \times l_m \times t + \mu \times R_s \times \sin\theta$

Where, $R_s = \dfrac{\tau_0}{1-\mu\left(\dfrac{h}{L}\right)} . d_m . t$

Substituting the values in above equation,

$R_s = 1057.5$ KN

$\therefore$ $V_f = 0.04\times24\times5.8\times1000\times200+0.3\times1057.5\times10^3\times\sin(24.145^0)$

$= 1113.6\times10^3 + 129.773\times10^3$

$= 1243.33\times10^3$N

$= 1243.33$ KN

(ii) $V_f = R_S \cos\theta$

$= 1057.5\times\cos 24.145^0$

$= 964.91$ KN

(iii) Failure shear force $V_i = 2/h_e(M_{ct} + M_{cc})_i + V_B$

Where,

$M_{ct} = 212.7$ KN×m

$M_{cc} = 343.0$ KN×m

$\therefore$ $V_i = 2\times(343.0+212.7)/1.55$

$= 717$ KN

For compression failure of diagonal strut

$$R_c = 0.67.Z.t.f'_m.\sec\theta$$

Where, $$Z = \frac{\pi}{2}\left[\frac{4E_c.I_g.h_m}{E_m.t.\sin 2\theta}\right]^{1/4}$$

E_c and I_g = Modulus of elasticity and M. I. of column sections respectively.

E_m and h_m = Modulus of elasticity and height of infill masonry.

θ = Angle between diagonal strut and horizontal.

Now substituting values in expression for Z we get,

$$Z = \frac{\pi}{2}\left[\frac{4\times 5700\sqrt{25}\times\frac{(700)^4}{12}\times 2.6}{600\times 24\times 200\times \sin 2\times 24.1455}\right]^{1/4}$$

$Z = 359.98$ mm

$\therefore$ $R_c = 0.67\times359.98\times200\times24\times\sec24.1455$

$$R_c = 1268.70\times10^3 \text{ N}$$

$$= 1268.70 \text{ KN}$$

The compressive diagonal force in panel should not be greater than R_c.

Stresses obtained from analysis for ground floor panel in frame in x-direction are as follows:

$F_{XY} = 620.84$ KN/m^2

$F_X = 2784.06$ KN/m^2

$F_Y = 447.64$ KN/m^2

$T_{max} = 1336.81$ KN/m^2

Principal stresses along diagonal $S_{max} = 2936.97$ KN/m^2

Maximum compressive stress carrying capacity of strut is:

$$= 1268.70/(0.20\times1.6)$$

$$= 3964.68 \text{ KN/m}^2$$

The compressive strength of masonry strut is greater than maximum principal stress in diagonal strut hence panel is safe.

Fig. A.2 (a) show some common failures observed in R.C. Framed buildings due to masonry infill walls. The notations used in above calculations for masonry infill are shown in Fig. A.2(b).