SEISMIC EVALUATION OF UN-REINFORCED MASONARY HOSPITAL BUIDLING

(MAJOR PROJECT REPORT - II)

For partial fulfilment of the requirements
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CERTIFICATE

This is to certify that the project titled **"SEISMIC EVALUATION OF UN-REINFORCED MASONARY HOSPITAL BUIDLING"** is a bonafide dissertation work carried out by me, Mohan Singh Yadav, Roll No. 2K11/STE/07, student of Master of Technology in Structures (Civil Engineering) from Delhi Technological University, New Delhi, during the session 2015-2016 towards the partial fulfilment of the requirements for award of the degree of Master of Technology in Structures (Civil Engineering).

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ABSTRACT

KEYWORDS: *Seismic evaluation, un-reinforced masonry, demand-to-capacity ratio, pushover analysis, plastic hinge, shear stress.*

It is well known that masonry buildings suffer a great deal of damage during earthquakes, leading to significant loss of lives. Almost 75% of the fatalities, attributed to earthquake in last century, is caused by collapse of buildings of which the greatest portion (more than 70%) is due to collapse of masonry buildings. A majority of the tenements in India are Unreinforced Masonry (URM) buildings that are weak and vulnerable even under moderate earthquakes. On the other hand, a cursory glance through the literature on earthquake resistant structures reveals that a bulk of research efforts is on RC structures. Clearly there is a great need to expend more effort in understanding masonry buildings subjected to earthquake induced dynamic loads.

The main aim of this thesis is to study the methodology given in various masonry structure related IS codes such as IS 1905-1987, IS 4326-1992 etc. for seismic evaluation of an more than 40 yrs old hospital building made of stone masonry. In hospital buildings more emphasis is given to non structural elements as they are part of important life line services therefore consideration of NSE is also incorporated. We have done firstly preliminary survey which include geometric properties of building as per ground and as per ledger, visual inspection and interaction with user and surrounding area. Then secondly we have gone for detailed seismic evaluation as per IS 1893 : 2002 & IS 1905 : 1987 for various failure mechanisms both local & global. Thirdly, we have suggested retrofitting measures as per various IS codes and also done cost comparison of retrofitting with respect to new construction. Lastly we have suggested some more measure keeping in view criticality of hospital.

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ABBREVIATIONS

URM	-	Un-Reinforced Masonry
NSE	-	Non Structural Element
IS	-	Indian Standard
EQ	-	Earthquake
ATC	-	Applied Technology Council
CQC	-	Complete Quadratic Combination
CSM	-	Capacity Spectrum Method
DCM	-	Displacement Coefficient Method
FEMA	-	Federal Emergency Management Agency
IS	-	Indian Standard
PGA	-	Peak Ground Acceleration
RC	-	Reinforced Concrete
SAP	-	Structural Analysis Program
SDOF	-	Single Degree of Freedom
FE	-	Finite Element
SRSS	-	Square Root of Sum of Square
DCR	-	Demand to Capacity Ratio
ADRS	-	Acceleration Displacement Response Spectrum
ISMC	-	Indian Standard Medium Channel
MCE	-	Maximum Considered Earthquake
DBE	-	Design Basis Earthquake

NOTATION

English Symbols

a	-	regression constant
c	-	classical damping
$C0$	-	factor for MDOF displacement
$C1$	-	factor for inelastic displacement
$C2$	-	factor for strength and stiffness degradation
$C3$	-	factor for geometric nonlinearity
d	-	effective depth of the section
db	-	diameter of the longitudinal bar
dp	-	spectral displacement corresponding to performance point
D	-	overall depth of the beam.
$D_n(t)$	-	displacement response for an equivalent SDOF system,
Ec	-	short-term modulus of elasticity of concrete
E_D	-	energy dissipated by damping
Es	-	modulus of elasticity of steel rebar
ES	-	maximum strain energy
E_{sec}	-	elastic secant modulus
EI	-	flexural rigidity of beam
fc	-	concrete compressive stress
f_{cc}	-	compressive strength of confined concrete
f_{co}	-	unconfined compressive strength of concrete
f_{ck}	-	characteristic compressive strength of concrete
Fe	-	elastic strength
f_y	-	yield stress of steel rebar
F_y	-	defines the yield strength capacity of the SDOF
f_{yh}	-	grade of the stirrup reinforcement
G	-	shear modulus of the reinforced concrete section
h	-	overall building height (in m)
k	-	lateral stiffness
k_e	-	confinement effectiveness coefficient
K_{eq}	-	equivalent stiffness

K_i - initial stiffness

l - length of frame element

l_p - equivalent length of plastic hinge

m - storey mass

M_n - modal mass for *n*th mode

N - number of modes considered

$P_{eff}(t)$ - effective earthquake force

$q_n(t$ - the modal coordinate for *n*th mode

R - Regular frame considered for study

$\{s\}$ - height-wise distribution of effective earthquake force

S_a - spectral acceleration

S_d - spectral displacement

CHAPTER 1
INTRODUCTION

1.1 BACKGROUND AND MOTIVATION

It is well known that masonry buildings suffer a great deal of damage during earthquakes. This is especially true for the unreinforced masonry (URM) buildings built in rural and semi-urban areas of developing countries. Fig. 1.1 shows a typical load bearing URM building. Many heritage buildings around the world are of old and thick walled masonry. Their value, historic, artistic, social or financial, is great and damage to them in an earthquake involves very costly repair.

Fig.1.1: Typical load bearing masonry construction for a residential building

Normally thick walled URM buildings were designed for vertical loads, since masonry has adequate compressive strength the structure behaves well as long as the loads are vertical. When such a masonry structure is subjected to lateral inertial loads during an earthquake, the walls develop shear and flexural stresses. The strength of masonry under these conditions often depends on the bond between brick and mortar. A masonry wall can also undergo in-plane shear stresses if the lateral forces are in the plane of the wall. Shear failure in the form of diagonal cracks is observed due to this. However, catastrophic collapses take place when the wall experiences out-of-plane flexure. This can bring down a roof and cause more damage. Fig. 1.2 shows typical failure of an URM building during 2010 Haiti earthquake.

Fig.1.2: Failure of an URM building during 2010 Haiti earthquake

Masonry buildings with light roof such as tiled roof are more vulnerable to out-of-plane vibrations since the top edge can undergo large deformations, due to lack of lateral restraint. Damage to masonry buildings in earthquakes may be influenced by four general categories: quality of materials and construction, connections between structural elements, structural layout and soil-structure interaction.

1.2 OBJECTIVE OF THE THESIS

Based on the literature review presented in Chapter 2 the salient objective of this research is defined as:

To do sesmic evaluation of an unreinforced masonry hospital building located in sesmic zone 5 with respect to structural and non structural element both and suggest retrofitting measures if any along with there rate analysis as compared to new contruction.

1.3 SCOPE OF THE STUDY

Most of the old buildings are masonry in nature which does not have seismic provisions as compared to new r.c.c. framed structure for which many IS codes are there to incorporate seismic resistance. Therefore it is very necessary to do seismic analysis and suggest corresponding retrofitting measures to ensure minimum damage to old buildings which are most vulnerable in case of natural disasters.

1.4 METHODOLOGY

The steps undertaken in the present study to achieve the above-mentioned objectives are as follows:

a) Carry out extensive literature review, to establish the objectives of the research work.

b) Preliminary survey of site such as length and breadth of building, year of construction, type of contruction, visual inspection, etc.

c) Detailed sesmic analysis as per is 1893:2002 with linear static procedure has been carried out for masonry structure with the help of IS codes 1905:1987, IS 4326:1992, IS 13935”

d) To check individual structural members and provide with retrofitting measures if necessary.

e) Comparisons of rate of retrofitting measures as with that of new construction. Based on DSR (Delhi Schedule of Rates).

1.5 ORGANISATION OF THE THESIS

This introductory chapter has presented the background, objective, scope and methodology of the present study. Chapter 2 starts with a description of the previous work done on unreinforced masonry wall by other researchers.

Chapter 3 deals with the case study for seismic evaluation & corresponding retrofitting measures.

Finally, Chapter 4 presents a summary including salient features, significant conclusions from this study and the future scope of research in this area.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

The first half of this chapter is devoted to a review of published literature on unreinforced masonry (URM) buildings. This part describes a number of experimental and analytical works on unreinforced masonry buildings.

The second half of this chapter is devoted to a review of seismic evaluation methods available in literature. This includes different evaluation methods based on linear and nonlinear analyses.

There are a number of research papers and design guidelines found on the structural properties of unreinforced masonry buildings

A number of studies were carried out by Jai Krishna and Chandra (1965) and Jai Krishna *et. al.* (1966). They studied the static in-plane strength of walls with and without reinforcement. They carried out the building analysis by considering the shear walls alone, with different parameters such as the aspect ratio of shear walls and size and location of openings in shear walls.

Arioglu and Anadol (1973) refer to the several earthquakes in Turkey and point out that plain masonry buildings are most vulnerable to earthquake damage. They refer to the special indigenous technique of producing horizontal wooden reinforcement on both faces at some vertical intervals to prevent collapse of masonry structures. Such practices have been traditionally in vogue in Turkey.

Abrams (1992) examines the in-plane lateral load behaviour of un-reinforced masonry elements under monotonic and cyclic loading. He argues that although masonry is considered to be brittle it has considerable deformation capacity after the development of first crack. Several suggestions have been made to evaluate the masonry strength characteristics under seismic loading.

Bruneau (1994) makes a number of observations on the seismic performance of un-reinforced masonry buildings (URM). Some of the types of failures are listed as

a) Lack of anchorage between floor and walls

b) Anchor failure when joists are anchored to walls

c) In-plane failure

d) Out-of-plane failure

e) Combined in-plane

Among these he emphasis that URM buildings are most vulnerable to flexural our-of-plane failure. In-plane failure may not right away lead to collapse since the load carrying capacity of a wall is not completely lost by diagonal cracking. However, our-of-plane failure leads to unstable and explosive collapse. Sometimes an initial in-plane failure may weaken the wall and subsequent out-of-plane motion can lead to collapse.

Rai and Goel (1996) also studied the seismic strengthening of un-reinforced masonry piers with steel elements. They considered the in-plane behaviour of masonry piers. The strengthening system showed significant improvement in stiffness and ductility.

Scrivener (1996) has done a survey of the damage to old masonry buildings in earthquakes around the world. He also reported the cause of the damage under four headings: quality of materials and construction, connections between structural elements, structural layout and soilstructure interaction.

Tomazevic (1999) and his colleagues carried out a large number of Earthquake Resistant Masonry Structures. He has discussed a number of concepts for designing earthquake resistant masonry and for retrofitting partially damaged masonry structures. The following concepts may be mentioned;

a) Traditional stone masonry walls with horizontal RC bond beams connecting the walls around the building at vertical spacing of 1.0 m or 2.0 m depending on the expected seismic intensity.

b) Masonry confined in its own plane by RC bond beams and columns. The columns have to be connected to the walls through shear keys. The spacing of columns is not more than 4.0 m.

c) Vertical reinforcement is provided n grouted holes of hollow block masonry and small pockets inside brick masonry. Horizontal reinforcements in the shape of truss like arrangements are also provided in bed joints. There are Euro code specifications for such reinforcements.

d) Horizontal tie rods are provided as a retrofitting measure in grooves cut in the mortar, below the floor level, on both sides of a wall. They are anchored to steel plates at both ends of the wall.

A steel mesh is anchored to the walls on the faces and covered with plaster.

A report by Navalli (2001) refers to the practice Uttaranchal where they use horizontal timber bands at different level improve the integrity of the masonry structure. Such houses suffered little damage during the October 1991 Uttarkashi earthquake. The paper by Jai Krishna and Arya (1962) also refers to such practices.

This section, however, discusses the previous research work on the lateral load behaviour of URM buildings. Andreas *et. al.* (2002) discussed the analysis of un-reinforced masonry buildings, and also discussed, and under what conditions, a simple equivalent frame model can be used for assessment purposes. Several parametric analyses involving finite element (FE) models of two-dimensional and three-dimensional structures have been performed in the elastic range, using both refined and coarse planar meshes.

Bulk of publication on earthquake resistance of structures deals with RC structures. There have been quite a few publications on earthquake resistant of masonry structures, from different parts of the world. A representative list of publications on such masonry is discussed here. Unreinforced Masonry Buildings and Earthquakes (FEMA P-774) described the risk assessment and guidelines how to minimise the risk of failure for existing URM Building in the year 2009 in California.

Bilgin and Korini (2012) examined the reason and capacity to failure by earthquake at Albania for the pre-defined template residential building. They carried out mainly three template building and analysed accordingly to ASC guideline.

2.2 SEISMIC EVALUATION METHODS

The following are the methods recommended for detailed seismic evaluation of buildings: (i) Linear static analysis – Equivalent static analysis, (ii) Linear dynamic analysis – Response spectrum analysis and (iii) Non-linear static analysis – Push-over analysis. It is recommended that all the above methods be performed sequentially for a proper assessment of the seismic vulnerability in a building. It may be noted that more rigorous analysis (nonlinear dynamic timehistory analysis) is possible, but this is not recommended as it is more involved and time consuming and not recommended for normal building. This section briefly explains the linear static and linear dynamic analyses as recommended in Indian Standard IS 1893: 2002. The main purpose of these analyses, from the seismic evaluation perspective, is to check the demand-tocapacity ratios of the building components and thereby ascertain code compliance. The two different linear analysis methods recommended in IS 1893: 2002 are explained in this Section. Any one of these methods can be used to calculate the expected seismic demands on the lateral load resisting elements.

2.2.1 Equivalent Static Method

In the equivalent static method, the lateral force equivalent to the design basis earthquake is applied statically. The equivalent lateral forces at each storey level are applied at the floor level. The base shear ($V = V_B$) is calculated as per Clause 7.5.3 of IS 1893: 2002.

$$V_B = A_h W \qquad (2.1)$$

$$A_h = \left(\frac{Z}{2}\right) \frac{I}{R} \frac{S_a}{g} \qquad (2.2)$$

where W= seismic weight of the building, Z= zone factor, I = importance factor,

R = response reduction factor, S_a /g= spectral acceleration coefficient determined from Fig. 2.1, corresponding to an approximate time period (T_a) which is given by

$T_a = 0.075h^{0.75}$ for RC moment resisting frame without masonry infill (2.3a)

$T_a = \dfrac{0.09h}{\sqrt{d}}$ for RC moment resisting frame with masonry infill (2.3b)

The base dimension of the building at the plinth level along the direction of lateral forces is represented as d (in metres) and height of the building from the support is represented as h (in metres). The response spectra functions can be calculated as follows:

For Type I soil (rock or hard soil sites): $$\frac{S_a}{g} = \begin{cases} 1+15T & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.40 \\ \dfrac{1}{T} & 0.40 \le T \le 4.00 \end{cases}$$

For Type II soil (medium soil): $$\frac{S_a}{g} = \begin{cases} 1+15T & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.55 \\ \dfrac{1.36}{T} & 0.55 \le T \le 4.00 \end{cases}$$

For Type III soil (soft soil): $$\frac{S_a}{g} = \begin{cases} 1+15T & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.67 \\ \dfrac{1.67}{T} & 0.67 \le T \le 4.00 \end{cases}$$

The design base shear is to be distributed along the height of building as per Clause 7.7.1 of IS1893: 2002.

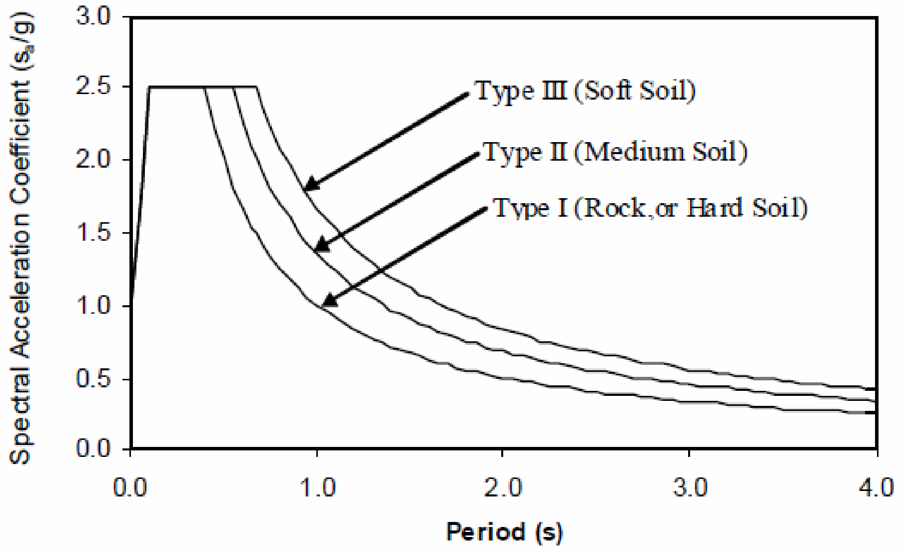


Fig. 2.1: Response spectra for 5 percent damping (IS 1893: 2002)

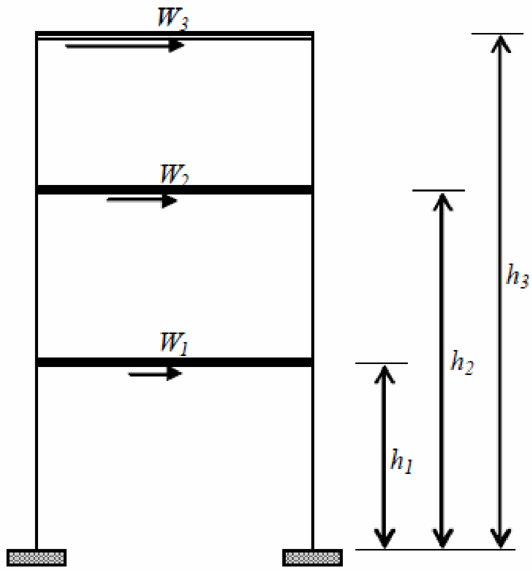


Fig. 2.2: Building model under seismic load

The design lateral force at floor *i* is given as follows

$$Q_i = V_B \; \frac{\mathrm{W}_i \; h_i^{\,2}}{\sum_{j=1}^{n} W_i h_1^{\,2}}$$

Here W_i=Seismic weight of floor i, h_i =Height of floor measured from base,

n= Number of storeys in the building equal to the number of levels at which masses are located (Fig. 2.2).

2.2.2 Response spectrum analysis

The equations of motion associated with the response of a structure to ground motion are given by:

$$\mathbf{M}\ddot{\mathbf{u}}(t) + \mathbf{C}\dot{\mathbf{u}}(t) + \mathbf{K}\mathbf{u}(t) = \mathbf{m}_{\mathbf{x}}\ddot{u}_{gx}(t) + \mathbf{m}_{\mathbf{x}}\ddot{u}_{gy}(t) + \mathbf{m}_{\mathbf{x}}\ddot{u}_{gz}(t)$$

Here, **M** is the diagonal mass matrix, **C** is the proportional damping matrix, **K** is the stiffness matrix, $\ddot{\mathbf{u}}$, $\dot{\mathbf{u}}$ and $\mathbf{u}$ are the relative (with respect to the ground) acceleration, velocity and displacement vectors, respectively, $\mathbf{m_x}$, $\mathbf{m_y}$, and $\mathbf{m_z}$ are the unit acceleration loads and $\ddot{\mathbf{u}}_{gx}$, $\ddot{\mathbf{u}}_{gy}$ **and** $\ddot{\mathbf{u}}_{gz}$, are the components of uniform ground acceleration.

The objective of response spectrum analysis is to obtain the likely maximum response from these equations. The earthquake ground acceleration in each direction is given as a response spectrum curve. The response spectrum is a plot of the maximum response (maximum displacement, velocity, acceleration or any other quantity of interest) to a specified load function for all possible single degree-of-freedom systems. The abscissa of the spectrum is the natural period (or frequency) of the system and the ordinate is the maximum response. It is also a function of damping. Fig.2.1 shows the design response spectra given in IS 1893: 2002 for a 5% damped system. According to IS 1893: 2002, high rise and irregular buildings must be analysed by the response spectrum method. However, this method of linear dynamic analysis is also recommended for regular buildings.

Response spectrum analysis is performed using mode superposition, where free vibration modes are computed using eigenvalue analysis. The maximum modal response (λ_k) of a quantity (considering the mass participation factor) is obtained for each mode of all the modes considered. Sufficient modes (r) to capture at least 90% of the participating mass of the building (in each of the orthogonal horizontal directions), have to be considered in the analysis. The modal responses of all the individual modes are then combined together using either the square root of the sum of the squares (SRSS) method or complete quadratic combination (CQC) method. The SRSS method is based on probability theory and is expressed as follows.

$$\lambda = \sqrt{\sum_{k=1}^{r} (\lambda_k)^2}$$

If the building has very closely spaced modes then the CQC method is preferable.

The base shear is calculated for response spectrum analysis in the following manner. The S_a/g value corresponding to each period of all the considered modes is first calculated from Fig. 2.1.

The base shear corresponding to a mode is then calculated as per the design code. Each base shear is multiplied with the corresponding mass participation factor and then combined as per the selected mode combination method, to get the total base shear of the building.

If the base shear calculated from the response spectrum analysis (V'_B) is less than the design base shear (V_B) calculated from Equation 2.1, then as per IS 1893: 2002, all the response quantities (member forces, displacements, storey shears and base reactions) have to be scaled up by the factor V_B / V'_B.

2.2.3 Evaluation Results

The demands (moments, shears and axial forces) obtained at the critical sections from the linear analyses are compared with the capacities of the individual elements. The capacities of RC members are to be calculated as per IS 456: 2000. The demand-to-capacity ratio (DCR) for each element should be less than 1.0 for code compliance. For a beam, positive and negative bending moment demands at the face of the supports and the positive moment demands at the span need to be compared with the corresponding capacities. For a column, the moment demand due to biaxial bending under axial compression must be checked using the P-M_x-M_y surface (interaction surface), generated according to IS 456: 2000.

2.3 PUSHOVER ANALYSIS – AN OVERVIEW

The use of the nonlinear static analysis (pushover analysis) came in to practice in 1970's but the potential of the pushover analysis has been recognized for last 10-15 years. This procedure is mainly used to estimate the strength and drift capacity of existing structure and the seismic demand for this structure subjected to selected earthquake. This procedure can be used for checking the adequacy of new structural design as well. The effectiveness of pushover analysis and its computational simplicity brought this procedure in to several seismic guidelines (ATC 40, FEMA 356 and ASCE/SEI 41-06) and design codes (Eurocode 8 and PCM 3274) in last few years.

Pushover analysis is defined as an analysis wherein a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a 'target displacement' is exceeded. Target displacement is the maximum displacement (elastic plus inelastic) of the building at roof expected under selected earthquake ground motion. Pushover analysis assesses the structural performance by estimating the force and deformation capacity and seismic demand using a nonlinear static analysis algorithm. The seismic demand parameters are global displacements (at roof or any other reference point), storey drifts, storey forces, component deformation and component forces. The analysis accounts for geometrical nonlinearity, material inelasticity and the redistribution of internal forces. Response characteristics that can be obtained from the pushover analysis are summarised as follows:

a) Estimates of force and displacement capacities of the structure. Sequence of the member yielding and the progress of the overall capacity curve.

b) Estimates of force (axial, shear and moment) demands on potentially brittle elements and deformation demands on ductile elements.

c) Estimates of global displacement demand, corresponding inter-storey drifts and damages on structural and non-structural elements expected under the earthquake ground motion considered.

d) Sequences of the failure of elements and the consequent effect on the overall structural stability.

e) Identification of the critical regions, where the inelastic deformations are expected to be high and identification of strength irregularities (in plan or in elevation) of the building.

Pushover analysis delivers all these benefits for an additional computational effort (modelling nonlinearity and change in analysis algorithm) over the linear static analysis. Step by step procedure of pushover analysis is discussed next.

CHAPTER 3

SESMIC EVALUATION A CASE STUDY OF AN EXISTING UN-REINFORCED MASONRY BUILDING

3.1 INTRODUCTION

The District Hospital, Dharmshala in Kangra District of Himachal Pradesh is 300 bedded hospital. Apart from the casualty ward, on an average 352 patients visit the District Hospital on daily basis.

It constitutes of five blocks/buildings namely Block-A, Block-B, Block-C, Laboratory Block and Nurse Hostel wherein all hospital facilities including the administrative department are present. The Ramp is in another six storied building by itself and also acts as a medium for connection between Block-A, Block-B and Block-C which are in proximity of each other.

Block-A and Block-B are stone masonry structures constructed in year 1969 while Block-C and the ramp structure are RC Framed structures constructed in 1989. The Laboratory Block comprised of two structures. The old structure of stone masonry constructed around 1983 and a new structure in both stone masonry and brick masonry constructed at a later date. The Nurse hostel was also a stone masonry structure constructed around the same period as that of Block-A.

Block-A and Block-C were constructed on sloping terrain while Block-B, Laboratory Block and the ramp structure were all constructed on flat terrain.

The soil type informed by the local engineer was sandy with gravel with bearing capacity of $15t/m^2$ at a depth of 1.5meters below ground level. The depth of water table on average basis was informed as 60meters below ground level.

Dharmshala lies in seismic zone V as per IS1893:2002 and as the hospital buildings in consideration have an importance factor I=1.5, hence as per IS4326:1993, all blocks fall under the most critical i.e. ‘E’ category.

Block-A

Block-B

Part of Block-C

Laboratory Block & Nurse Hostel in lower left hand corner

3.2 SITE OBSERVATIONS

LOAD BEARING / MASONRY STRUCTURES

BLOCK NAME	BLOCK-A
YEAR, TYPE OF STRUCTURE	1969, Load Bearing Structure.
TOTAL FLOOR AREA	3862.50 m2
FACILITIES PRESENT	Zonal Medical Store, District Medical Store, X-Ray rooms, Operation Theater & ICU, Administrative Block, ENT, Lecture Theater, Orthopedic, dental clinic, etc.
CONFIGURATION IRREGULARITIES (PLAN & VERTICAL) 	Vertical geometric Irregularity was observed. The entire structure is G+2 storeys, but in one end of the block it is B+G+2 making it more than 3 storeys. IS Code does not permit more than 3 storeys for 'E' category buildings which makes the present structure very vulnerable and hence can be categorised as 'E+' category. This can also cause Torsional irregularity in the building structure.
FOUNDATION DETAILS	Step foundation in stone masonry of width 1.0 meter and depth 1.05 meters below ground level.

PLINTH BAND / BEAM	Plinth band of 400mm width and 150mm depth provided in exterior longitudinal walls only making it ineffective to tie the structure as per IS4326:1993.
WALL / INFILL PANEL  	Dressed Stone Masonry walls constituting of multi wythes and having total thickness of 400mm. General ratio of openings found in walls was found to range between 0.4 - 0.9 making it unsafe as per IS4326:1993.
LINTEL BAND / BEAM	Continuous lintel beam of 400mm width and 150mm depth provided in exterior longitudinal walls only, making it ineffective to tie the structure as per IS4326:1993.
INTERMMEDIATE ROOF / FLOOR	All intermediate Floors are cast in-situ RCC Floors of 150mm thickness, simply rested on the RCC beams / Stone masonry walls as applicability.
ROOF / EAVE LEVEL BAND / BEAM	Not observed. To be provided as per IS4326:1993.
GABLE BAND / BEAM	Not observed. To be provided as per IS4326:1993.

ROOF DETAILS i/c CONNECTIONS, ETC. 	Pitched roof comprising of Kingpost Wooden truss @ 3.0meters center to center with asbestos sheeting above was found at site. The purlin & rafter sections of 80mmX150mm and tie, struts & inclined members of size 80mmX125mm were measured. The Trusses were not fixed to the wall. There was no bracing between the trusses in horizontal or inclined pane. This makes it more vulnerable as per criteria given in IS4326:1993.
OTHER STRUCTURAL FEATURES OBSERVED	
Expansion Joint 	The dimensions of Block-A is 82.6X15meters. Two expansion joints of 40mm each were observed perpendicular to the length of building, dividing the structure into three lengths. The end parts are each of length of 25.4meters while the middle part is of length 31.4meters. Though the expansion joints were clear in the walls, but they had been compromised with continuous floor finish.
Mezzanine Floor	RCC Roof cast at a later date above the Toilet zone in Operation Theater Portion to house 8 nos. 1000lts. Sintex Water Tanks.
DISTRESS OBSERVATIONS	

Dampness  	Observed in many areas near ducts and internal face of exterior walls where water drainage pipelines were attached.
Plaster Chip-off	Observed in damp areas.
Cracks 	Separation of the longitudinal wall and cross wall was observed in Post OT room suggesting no proper connection between longitudinal wall and cross walls. Moreover it was informed by the OTA that such cracks had come in many walls but they were filled up during renovation.
Termites	Observed in corner of passage way in OT zone.

BLOCK NAME	BLOCK-B
YEAR, TYPE OF STRUCTURE	1969, Load Bearing Structure.
TOTAL FLOOR AREA	840 m^2
FACILITIES PRESENT	Blood Bank, Imaging Department (Ultrasound & CT Scan), Gynaecological Dept, Minor OT - Labour Room, Gynaecology Ward, etc.
CONFIGURATION IRREGULARITIES (PLAN & VERTICAL)	No irregularity was observed.
FOUNDATION DETAILS	Step foundation in stone masonry of width 1.0 meter and depth 1.05 meters below ground level.
PLINTH BAND / BEAM	Not observed. As per IS4326:1993, the plinth band must be provided for better seismic performance.
WALL / INFILL PANEL 	Dressed Stone Masonry walls constituting of multi wythes and having total thickness of 400mm. Partition/Cross walls of Half Brick thickness i.e. 150mm were observed. General ratio of openings found in walls was found to range between 0.4 - 0.9 making it unsafe as per IS4326:1993.

LINTEL BAND / BEAM	Continuous Lintel Beam of size 400mmX150mm depth provided along exterior longitudinal walls for ground floor only making it ineffective to tie the structure as per IS4326:1993.
INTERMMEDIATE ROOF / FLOOR	All intermediate Floors are cast in-situ RCC Floors of 150mm thickness, simply rested on the RCC beams / Stone masonry walls as applicability.
ROOF / EAVE LEVEL BAND / BEAM	Not observed. To be provided as per IS4326:1993.
GABLE BAND / BEAM	Not observed. To be provided as per IS4326:1993.
ROOF DETAILS i/c CONNECTIONS, ETC. 	The First Storey had a false ceiling above which was a pitched roof made of Wooden Truss like members (The Truss was in two separate parts being disjointed at the tie level and at the ridge level) placed at the center to center distance of 3.0 meters with purlin sections of 100mmX160mm and rafters, tie, struts & inclined members of size 80mmX100mm.Steel Flats were welded and bolted to connect the wooden members. This connection was improper and needs to be replaced with proper joint system. Bracing in lateral plane and inclined plane were absent. This made it very vulnerable under seismic activity. Asbestos sheeting was used as sheeting material. Many of the asbestos sheets were damaged and

	water was pouring inside the building.
OTHER STRUCTURAL FEATURES OBSERVED	None Observed.
DISTRESS OBSERVATIONS	
Dampness 	In almost all rooms due to cracked Asbestos sheet roofing on top floor, dampness and Moss generation was observed on the walls and bottom of roof.

<table>
<tr><td>Plaster Chip-Offs
</td><td>Due to Dampness Plaster had peeled off in few locations.</td></tr>
<tr><td>Termites
</td><td>Termites were also observed on damp areas on walls.</td></tr>
<tr><td></td><td></td></tr>
</table>

BLOCK NAME	LABORATORY BLOCK
YEAR, TYPE OF STRUCTURE	1983, Load Bearing Structure.
TOTAL FLOOR AREA	665 m^2
FACILITIES PRESENT	All labs related to pathology i.e. aids, tuberculosis, biochemistry lab, etc. are present in the entire block.
CONFIGURATION IRREGULARITIES (PLAN & VERTICAL)	Vertical irregularity was observed due to basement toilet at one end of the old block. This can also result in creating Torsional irregularity.
FOUNDATION DETAILS	Foundation is stripped foundation.
PLINTH BAND / BEAM	Not observed. As per IS4326:1993, the plinth band must be provided for better seismic performance.
WALL / INFILL PANEL 	In Old Block walls are stone masonry in cement mortar in two separate wythes of total thickness of 400mm. All interior walls also made of Stone Masonry similar to exterior walls. In New Block Stone Masonry Wall in cement Mortar of thickness 400mm was observed. All internal walls were constructed of Brick masonry of thickness 250mm. Ratio of openings was calculated to be ranging between 0.4 -0.9 which is much higher than limited as per IS code.

LINTEL BAND / BEAM	Continuous in new block on both floors in longitudinal exterior walls of size 400mmX150mm depth making it ineffective to tie the structure as per IS4326:1993.
INTERMMEDIATE ROOF / FLOOR	Intermediate Floor are cast in-situ RCC Floors of 150mm thickness, simply rested on the RCC beams / Stone masonry and brick walls as applicability.
ROOF / EAVE LEVEL BAND / BEAM	Not observed. To be provided as per IS4326:1993.
GABLE BAND / BEAM 	Observed only in one end of new block. In old Block this was absent. To be provided as per IS4326:1993.
ROOF DETAILS i/c CONNECTIONS, ETC. 	The first Storey had a false ceiling above which was a sloping roof. The arrangement observed was C.G.I sheeting fixed over Steel Truss in new Block. In Old Block the original RCC roof existed, but due to leakage in due course of time a steel truss roof was erected over the RCC Roof. Details could not be verified due to inaccessibility.

OTHER STRUCTURAL FEATURES OBSERVED	None Observed.
DISTRESS OBSERVATIONS	
Dampness 	Dampness observed on exterior walls and underneath water tank over corridor connecting old and new block.
Cracks 	Structural crack due to settlement observed in store and bathroom in corner of old block. This is likely due to washing away of soil beneath the Water Closet located at basement level. Cracks also observed on RCC Columns in corridor under Steel Water Tank. Numerous cracks observed in all rooms coming from false ceiling towards lintel level.

RC FRAMED STRUCTURES

BLOCK NAME	BLOCK-C
YEAR, TYPE OF STRUCTURE	1989, RC Framed Structure.
TOTAL FLOOR AREA	7000 m^2
FACILITIES PRESENT	Casualty, Medical Ward,

	Gynaelogical Ward, Minor OT, Child Ward, Kitchen, etc.
CONFIGURATION IRREGULARITIES (PLAN & VERTICAL) 	Vertical irregularity was observed due to double basement at one end of the block. This can also result in creating Torsional irregularity. Re-Entrant corners were also observed at the basement end of the block. The block had a Non-Parallel system arrangement of columns.
FOUNDATION DETAILS	Isolated Foundation.
PLINTH BAND / BEAM	Not observed.
WALL / INFILL PANEL 	Exterior walls comprising of 50mm thick concrete panel with 150mm stone fascia cladding making it total of 200mm thick exterior walls. Interior walls of brickwork generally 250mm thick with partition walls of thickness 130mm at few locations in cement mortar was observed. Partition wall of 1.0 meter height and 130mm thickness was also observed in Wards having unsupported length of 7.0 meters. Ratio of openings was found 0.4 to 0.9 which is greater than the limit mentioned in the IS codes.
LINTEL BAND / BEAM	Present and continuous on long exterior walls of size 150mmX200mm with projecting sun shades.
INTERMMEDIATE ROOF / FLOOR	Intermediate Floors are cast in-situ RCC Floors of 150mm thickness, simply rested on the RCC beams.

ROOF / EAVE LEVEL BAND / BEAM	Eave level Band provided.
GABLE BAND / BEAM	Absent.
ROOF DETAILS i/c CONNECTIONS, ETC. 	The third Storey had a false ceiling above which was a pitched roof. The arrangement observed was C.G.I sheeting fixed over Angle section purlins and rafters rested on Slender RCC Columns at intervals of 115mm c/c.
OTHER STRUCTURAL FEATURES OBSERVED	
Expansion Joint 	Two expansion joints of 20mm each were observed perpendicular to the length of building, dividing the structure into three lengths.
Mezzanine Floor	RCC Roof cast above the Toilet zone in Operation Theater Portion to house 8 nos. 1000lts.

	
Dampness 	Dampness observed near all ducts housing water drain pipelines, etc. Moss generation was also seen in these locations.
Plaster Chip-Offs 	Observed in almost all places where dampness had occurred. Especially in the Electrical room in basement 2 of the building.

3.3 NON-STRUCTURAL COMPONENTS/CONTENTS/EQUIPMENTS/ ELEMENTS

<table>
<tr><td>NON-STRUCTURAL ELEMENTS</td><td>WATER SUPPLY SYSTEM</td></tr>
<tr><td>1- Waters tanks</td><td></td></tr>
<tr><td>Roof Top Water Tank Details:
</td><td>BLOCK A

8nos. Sintex Water Tanks each of 1000lts. are placed on Steel Frame resting on newly constructed RCC roof over Toilets in Operation Theatre zone. Sway and overturning of Four nos. of Sintex tanks is prevented by a Steel Channel Section fixed in the Brick wall provided above RCC roof. The remaining 4 nos. Sintex Tanks need to be protected from sway or overturning effect by proper anchoring with structural members.</td></tr>
<tr><td></td><td>BLOCK C

8nos. in groups of two steel water tanks of size 1.4mX1.4mX2.27m water tank placed over ISMB250 sections which rest on RCC Floor of 100mm thickness over toilets on topmost floor. Water tanks are not fixed and can be likely cause of danger in case of seismic activity. The water tanks need to be either relocated or anchored properly to prevent any sway or overturning during seismic activity.</td></tr>
</table>

	LABORATORY BLOCK Two number steel water tanks placed over RCC roof over Corridor. Water tank not fixed and can be likely cause of danger in case of seismic activity.
2- Pipelines	
Utility pipelines details: 	Water drainage pipelines were observed within RCC Column.
	MEDICAL FACILITIES
3- Cabinets	
Storage units of Storage Cabinets, Computers, etc.: 	Not fixed to the Wall or anchored to any structural member, in case of seismic activity likely to overturn. In few places almirahs were placed in the middle of the room making it more dangerous. Also in administrative block rooms, the furniture is so placed that there is minimal space for the person inside the room to move efficiently.

	MEDICALS EQUIPMENT
4- Ot lights ,oxygen cylinder etc	
Layout of Medical Lab and Medical unit Equipment in rooms such as OT lights, autoclave machine, blood bank refrigerator, OT equipment oxygen cylinder, etc.: 	In OT, the heavy lights were clamped to steel pipe resting on the RCC beams / Stone masonry walls (as applicable). The Equipment in general was freely standing in the center of the room or placed against the walls. In other places the equipment was placed over the slab projecting from the wall. The equipment was lying freely which are likely cause of creating hazardous situation.
5- Layout of beds	
Layout of beds and equipment in ward rooms: 	Equipment with beds not fixed to the walls or any attachment with bed i.e. they were in loose sate. The equipment need to be anchored as per Clause 7.13 in IS1893 (Part 1) Draft Code.
6- Condition of Passage Way	
	BLOCKA Few Cupboards were found placed against the wall in the passage way without fastening to the wall. Likely to fall and block passageway.

 	
 	BLOCK B The entire imaging dept had only one Door of clear width of 2.0m to escape during any emergency.
 	LABORATORY The passage way had unanchored almirahs blocking the path. Even on first floor corridor unanchored almirahs were found standing against the walls, thereby reducing the clear passageway.

	GENERAL ITEMS, ELECTRICAL & MECHANICAL EQUIPMENT
7- Air conditioners ,etc	
Mechanical and electrical equipments such as control and distribution panels, pumps, generators, communication control equipment, Air Conditioners, etc. 	All such equipments were found in unanchored /unsecured position. Air-Conditioners were placed on exterior walls most of them in Operation Theatre zone. They were resting on steel frame without any anchoring. They need to be anchored either vertically or horizontally as the case arises as per Clause 7.13 of IS1893 (Part 1) Draft version.
8- Electrical wirings & fittings	
Type & condition of electrical Wiring & Fittings, presence of hanging fans, bulbs, etc.: 	All wirings done internally. In wards the Tubes were found in hanging condition. They need to be fastened properly to prevent sway during seismic activity.

9- Geysers	
Falling Hazards such as geysers, stabilizers, window A.C., etc. 	Geysers were fixed onto the wall above lintel height in few rooms. They need to be anchored as per clause 7.13 of draft version of IS1893.
10- Fire fighting facilities	
	Fire Fighting System nor fire Extinguishers were observed. When provided, they need to be anchored as per guidelines given in Clause 7.13 of IS1893 (Part 1) Draft version.

3.4 3D MODEL & PLAN OF BUILDING

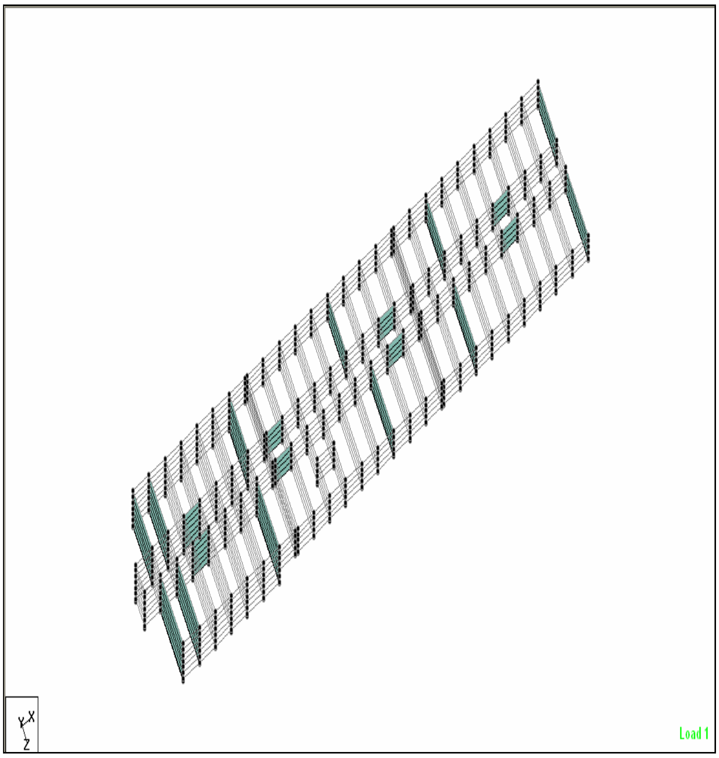


Fig. 3.1 (A) : 3D MODEL OF BUILDING

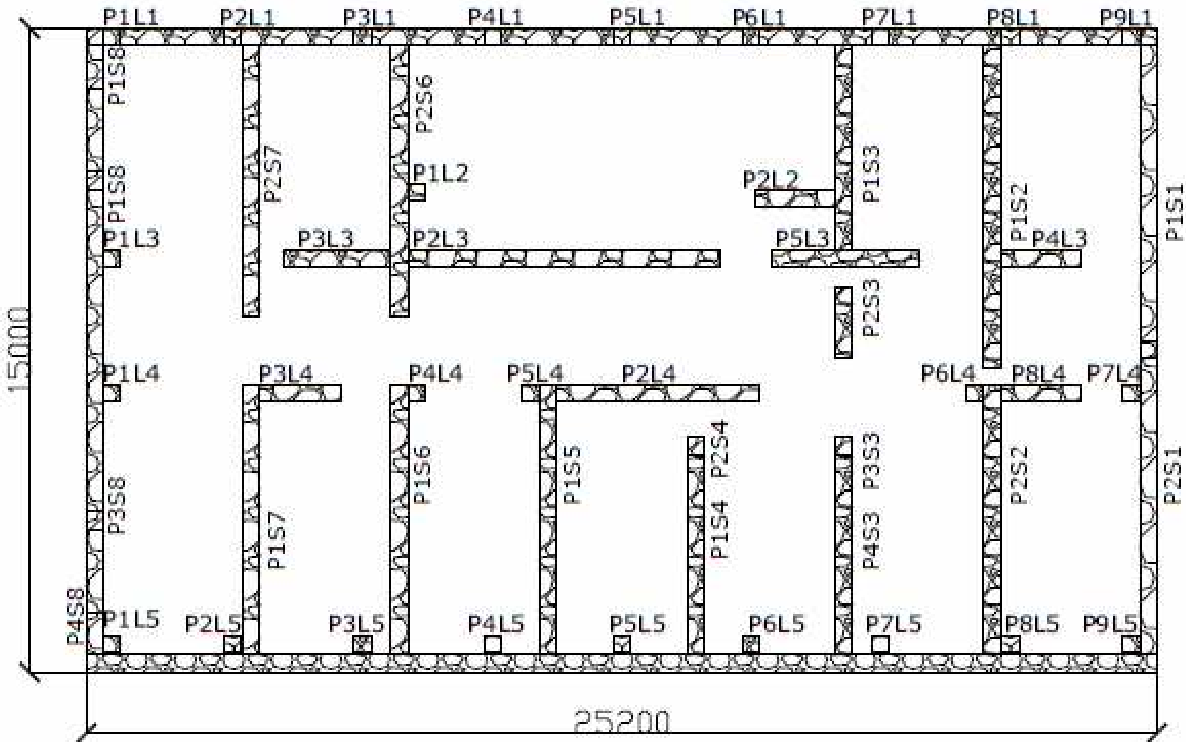


Fig. 3.2 (B) : PLAN OF BUILDING

3.5 DETAIL ANALYSIS REPORT OF MASONRY STRUCTURE

Masonry Structure Block A

GIVEN DATA

Length of building(L)	25.20	m
Breadth of building (B)	15.00	m
Thickness of walls (T)	0.40	m
Cast in-situ RCC slab thickness	0.15	m
Floor finish above slab	0.08	m

LOADING DATA (Loads taken from IS 875:1987(part1 & part2))

R.C.C =	26.00	KN /m³
Stone masonry =	22.00	KN /m³
Floor Finish =	24.00	KN /m³
Asbestos sheeting =	0.13	KN/m²
Floor dead load =	5.70	KN/m²
Floor Live load =	4.00	KN/m²
Unit wt of stone masonry per unit area	8.80	KN/m²

LOAD CALCULATION:

Roof dead load:	51.80 KN
Intermediate floor dead load:	2154.60 KN
Intermediate floor live load:	1512.00 KN
Due to wall load on roof level	2117.94 KN
Due to wall load on second floor level	4270.20 KN
Due to wall load on first floor	4297.26 KN
Due to wall load on ground floor	2775.96 KN

CALCULATION OF SEISMIC WEIGHT

As per clause 7.3.1 of IS 1893:2002

Imposed load to be considered in seismic weight

25% of imposed load =	1.00 KN/m²
Seismic weight on roof =	2169.74 KN
Seismic weight on second floor =	6802.80 KN
Seismic weight on first floor =	6829.86 KN
Seismic weight on ground floor =	5308.56 KN
Total seismic weight	21110.96 KN

CALCULATION OF BASE SHEAR

Z = 0.36 (seismic zone v)

I= 1.50 (Importance factor)

R= 2.50 (Response reduction factor: Since Horizontal Bands have already been provided)

H= 12.50 (Height of Building)

Ta= .09h/√d

When lateral force is perpendicular to width

Ta= 0.22 sec

When lateral force is perpendicular to width

Ta= 0.29 sec

Sa/g= 2.50

Sa/g= Average response acceleration

The distribution of shear force in the vertical direction is made as per IS: 1893:2002

V_B = ((Z X I)/(2 X R))X Sa/g x W = 5699.96 KN

DESIGN LATERAL FORCES AT EACH FLOOR LEVEL

Total height from basement level to roof level = 12.50 m

Total height from basement level to second floor level = 9.38 m

Total height from basement level to first floorlevel = 6.23 m

Total height from basement level to ground floor level = 3.08 m

LATERAL FORCE DISTRIBUTION
As per clause 7.7.1 of IS 1893:2002

Storey	Wi (kn)	hi (m)	Wihi² (KNm²)	Wihi²/ΣWihi²	Lateral force at level(KN)
Roof	2545.60	12.50	397750.00	0.28	1603.42
Second	7580.00	9.38	666210.94	0.47	2685.64
First	7611.75	6.23	294960.07	0.21	1189.05
GF	5820.00	3.08	55031.74	0.04	221.85
Σ			**1413952.74**		**5699.96**

Distribution of shears among different walls at ground floor level
Calculation of center of gravity-:

	Area (m²)	Distance of C.G (m)
L1	46.35	14.225
L2	8.42	10.450
L3	62.75	8.925
L4	38.79	6.075
L5	44.73	0.775
C_{gy}=	**7.85 m**	

	Area (m²)	Distance of C.G (m)
S1	45.08	25.00
S2	44.36	21.90
S3	40.04	18.80
S4	22.61	15.70
S5	25.13	12.60
S6	39.38	9.50
S7	39.38	6.40
S8	10.58	0.20
C_{gx}=	**15.57 m**	

Determination of Stiffness of Walls:

The piers are assumed top be fixed at both ends, therefore the following formula is usedto calculate stiffness:

$K_i = (E_m.t)/ ((h/l) ^3 + 3*(h/l))$

Where

$E_m = 550 X f_m$

From IS: 1905:1987, we get fm = 750000 N/m^2

Therefore, E_m = 412.5 MN/m^2

Table : Calculation of Stiffness of piers and walls in short walls:

Pier No.	Length l (m)	Height h (m)	C_i (m)	h/l	K_i (MN/m)	$K_i/\sum Ki$
P_1S_1	6.90	1.50		0.22	249.076	0.500
P_2S_1	6.90	1.50		0.22	249.076	0.500
S_1			25.00		**498.153**	
P_1S_2	7.50	2.10		0.28	191.426	0.546
P_2S_2	6.30	2.10		0.33	159.107	0.454
S_2			21.90		**350.533**	
P_1S_3	4.65	2.10		0.45	114.033	0.367
P_2S_3	1.65	2.10		1.27	28.062	0.090
P_3S_3	0.45	1.50		3.33	3.508	0.011
P_4S_3	4.65	1.50		0.32	164.784	0.531
S_3			18.80		**310.388**	
P_1S_4	4.65	2.10		0.45	114.033	0.988
P_2S_4	0.45	2.10		4.67	1.427	0.012
S_4			15.70		**115.460**	
P_1S_5	6.30	3.00		0.48	107.383	1.000
S_5			12.60		**107.383**	
P_1S_6	6.30	3.00		0.48	107.383	0.500
P_2S_6	6.30	3.00		0.48	107.383	0.500
S_6			9.50		**214.767**	
P_1S_7	6.30	3.00		0.48	107.383	0.500
P_2S_7	6.30	3.00		0.48	107.383	0.500
S_7			6.40		**214.767**	
P_1S_8	1.00	3.00		3.00	4.583	0.449
P_2S_8	0.45	3.00		6.67	0.522	0.051
P_3S_8	0.45	3.00		6.67	0.522	0.051
P_4S_8	1.00	3.00		3.00	4.583	0.449
S_8			0.20		**10.210**	

Location of C_s from the center of the end short wall:

C_{sx} = 17.87 m

Calculation of stiffness of piers and walls in Long walls:

Pier No.	Length L(m)	Height H(m)	C_i (m)	h/l	K_i (MN/m)	$K_i/\sum Ki$
P_1L_1	0.40	1.50		3.75	2.579	0.111
P_2L_1	0.40	1.50		3.75	2.579	0.111
P_3L_1	0.40	1.50		3.75	2.579	0.111
P_4L_1	0.40	1.50		3.75	2.579	0.111
P_5L_1	0.40	1.50		3.75	2.579	0.111
P_6L_1	0.40	1.50		3.75	2.579	0.111
P_7L_1	0.40	1.50		3.75	2.579	0.111
P_8L_1	0.40	1.50		3.75	2.579	0.111
P_9L_1	0.40	1.50		3.75	2.579	0.111
L_1			14.23		**23.209**	
P_1L_2	0.40	2.10		5.25	1.028	0.028
P_2L_2	1.90	2.10		1.11	35.362	0.972
L_2			10.45		**36.391**	
P_1L_3	0.40	2.10		5.25	1.028	0.002
P_2L_3	8.10	2.10		0.26	207.494	0.462
P_3L_3	3.50	2.10		0.60	81.845	0.182
P_4L_3	1.90	2.10		1.11	35.362	0.079
P_5L_3	5.00	2.10		0.42	123.680	0.275
L_3			8.93		**449.410**	
P_1L_4	0.40	2.10		5.25	1.028	0.005
P_2L_4	5.00	2.10		0.42	123.680	0.625
P_3L_4	1.90	2.10		1.11	35.362	0.179
P_4L_4	0.40	2.50		6.25	0.628	0.003
P_5L_4	0.40	2.50		6.25	0.628	0.003

P_6L_4	0.40	2.50		6.25	0.628	0.003
P_7L_4	0.40	2.50		6.25	0.628	0.003
P_8L_4	1.90	2.10		1.11	35.362	0.179
L_4			6.08		**197.943**	
P_1L_5	0.40	1.50		3.75	2.579	0.128
P_2L_5	0.40	1.50		3.75	2.579	0.128
P_3L_5	0.40	1.50		3.75	2.579	0.128
P_4L_5	0.40	1.50		3.75	2.579	0.128
P_5L_5	0.40	1.50		3.75	2.579	0.128
P_6L_5	0.40	2.10		5.25	1.028	0.051
P_7L_5	0.40	2.10		5.25	1.028	0.051
P_8L_5	0.40	1.50		3.75	2.579	0.128
P_9L_5	0.40	1.50		3.75	2.579	0.128
L_5			0.78		**20.108**	

Location of C_s from the center of the backside long wall:
C_{sy} = 8.17 m

C_{sx} = 17.87 m C_{sy} = 8.17 m
C_{Gx} = 15.57 m C_{Gy} = 7.85 m

As Per Clause 7.9.2 of IS: 1893 :(Part 1) 2002

Eccentricity, $e_{di} = 1.5*e_{si} + 0.05b_i$

When load is in direction parallel to short walls:
e_{di} =   4.70 m

When load is in direction parallel to long walls:
e_{di} =   1.23 m

So, torsional moment developed due to eccentricity

Parallel to Long Walls M_T = 7024.96 KNm

Parallel to short Walls M_T = 26797.72 KNm

3.6 RESULT & DISCUSSION

Table 3.1 : Distribution of Translational & Torsional Shears at Ground Floor when load is in parallel to long walls:

FORCE PERPENDICUAR TO LONG WALLS VB & MT = 49840.20

FORCE PERPENDICUAR TO SHORT WALLS VB & MT = 13065.49

Wall	K_{ix} (MN/m)	K_{iy} (MN/m)	D_x (m)	D_y (m)	$K_i * D$	$K_i * D^2$	$V=(K_i/\sum K_i) * V_B$ (KN)	$V_M=(K_i.D_y/\sum K_i.D_y^2) * M_T$ (KN)	$V_T=V_M+V$ (KN)	$V=(K_i/\sum K_i) * V_B$ (KN)	$V_M=(K_i.D_y/\sum K_i.D_y^2)* M_T$ (KN)	$V_T=V_M+V$ (KN)	Total Shear V_T
S_1	-	498.15	-	17.87	8900.70	159032.54	1558.72	-1092.84	1558.72	-	286.49	286.49	1845.20
S_2	-	350.53	-	4.03	1413.55	5700.27	1096.82	-173.56	1096.82	-	45.50	45.50	1142.31
S_3	-	310.39	-	0.93	289.46	269.95	971.20	-35.54	971.20	-	9.32	9.32	980.52
S_4	-	115.46	-	2.17	250.25	542.40	361.27	-30.73	361.27	-	8.05	8.05	369.33
S_5	-	107.38	-	5.27	565.63	2979.43	336.00	-69.45	336.00	-	18.21	18.21	354.21
S_6	-	214.77	-	8.37	1797.04	15036.61	672.00	-220.64	672.00	-	57.84	57.84	729.84
S_7	-	214.77	-	11.47	2462.82	28242.18	672.00	-302.39	672.00	-	79.27	79.27	751.27
S_8	-	10.21	-	17.67	180.38	3186.92	31.95	22.15	54.09	-	5.81	5.81	59.90
		1821.66				**214990.29**							
L_1	23.21	-	6.06	-	140.55	851.13	-	17.26	17.26	181.95	4.52	186.47	203.73
L_2	36.39	-	2.28	-	83.00	189.31	-	10.19	10.19	285.29	-2.67	285.29	295.48
L_3	449.41	-	0.76	-	339.67	256.72	-	41.70	41.70	3523.25	-10.93	3523.25	3564.96
L_4	197.94	-	2.09	-	414.53	868.11	-	50.90	50.90	1551.82	-13.34	1551.82	1602.72
L_5	20.11	-	7.39	-	148.68	1099.38	-	18.26	18.26	157.64	-4.79	157.64	175.90
	727.06					**3264.66**							
					$\sum$	**218254.95**							

Geometric Properties of the piers in short walls:

Pier No.	Length L (m)	Area (m^2)	C_i (m)	$(I_g)_i$ m^4)
P_1S_1	6.90	2.76	3.45	10.950
P_2S_1	6.90	2.76	3.45	10.950
P_1S_2	7.50	3.00	3.75	14.063
P_2S_2	6.30	2.52	3.15	8.335
P_1S_3	4.65	1.86	2.33	3.351
P_2S_3	1.65	0.66	0.83	0.150
P_3S_3	0.45	0.18	0.23	0.003
P_4S_3	4.65	1.86	2.33	3.351
P_1S_4	4.65	1.86	2.33	3.351
P_2S_4	0.45	0.18	0.23	0.003
P_1S_5	6.30	2.52	3.15	8.335
P_1S_6	6.30	2.52	3.15	8.335
P_2S_6	6.30	2.52	3.15	8.335
P_1S_7	6.30	2.52	3.15	8.335
P_2S_7	6.30	2.52	3.15	8.335
P_1S_8	1.00	0.40	0.50	0.033
P_2S_8	0.45	0.18	0.23	0.003
P_3S_8	0.45	0.18	0.23	0.003
P_4S_8	1.00	0.40	0.50	0.033

Geometric Properties of the piers in Long walls:

Pier No.	Length L (m)	Area (m^2)	C_i (m)	$(I_g)_i$ (m^4)
P_1L_1	0.40	0.16	0.20	0.002
P_2L_1	0.40	0.16	0.20	0.002
P_3L_1	0.40	0.16	0.20	0.002
P_4L_1	0.40	0.16	0.20	0.002
P_5L_1	0.40	0.16	0.20	0.002
P_6L_1	0.40	0.16	0.20	0.002
P_7L_1	0.40	0.16	0.20	0.002
P_8L_1	0.40	0.16	0.20	0.002
P_9L_1	0.40	0.16	0.20	0.002
P_1L_2	0.40	0.08	0.20	0.001
P_2L_2	1.90	0.38	0.95	0.114
P_1L_3	0.40	0.16	0.20	0.002
P_2L_3	8.10	3.24	4.05	17.715
P_3L_3	3.50	1.40	1.75	1.429
P_4L_3	1.90	0.76	0.95	0.229
P_5L_3	5.00	2.00	2.50	4.167
P_1L_4	0.40	0.16	0.20	0.002
P_2L_4	5.00	2.00	2.50	4.167
P_3L_4	1.90	0.76	0.95	0.229
P_4L_4	0.40	0.16	0.20	0.002
P_5L_4	0.40	0.16	0.20	0.002
P_6L_4	0.40	0.16	0.20	0.002
P_7L_4	0.40	0.16	0.20	0.002
P_8L_4	1.90	0.76	0.95	0.229

P_1L_5	0.40	0.16	0.20	0.002
P_2L_5	0.40	0.16	0.20	0.002
P_3L_5	0.40	0.16	0.20	0.002
P_4L_5	0.40	0.16	0.20	0.002
P_5L_5	0.40	0.16	0.20	0.002
P_6L_5	0.40	0.16	0.20	0.002
P_7L_5	0.40	0.16	0.20	0.002
P_8L_5	0.40	0.16	0.20	0.002
P_9L_5	0.40	0.16	0.20	0.002

Distribution of Lateral Shear among Piers in short walls:

Pier No.	$V_T (K_i/\sum K_i)$ (KN)	Total
P_1S_1	922.60	1845.20
P_2S_1	922.60	
P_1S_2	623.82	1142.31
P_2S_2	518.50	
P_1S_3	360.23	
P_2S_3	88.65	
P_3S_3	11.08	980.52
P_4S_3	520.55	
P_1S_4	364.76	369.33
P_2S_4	4.56	
		354.21
P_1S_5	354.21	
P_1S_6	364.92	729.84
P_2S_6	364.92	

P_1S_7	375.64	751.27
P_2S_7	375.64	
P_1S_8	26.89	
P_2S_8	3.06	59.90
P_3S_8	3.06	
P_4S_8	26.89	

Distribution of Lateral Shear among Piers in long walls:

Pier No.	$V_T\,(K_i/\sum K_i)$ (KN)	Total
P_1L_1	22.64	
P_2L_1	22.64	
P_3L_1	22.64	
P_4L_1	22.64	203.73
P_5L_1	22.64	
P_6L_1	22.64	
P_7L_1	22.64	295.48
P_8L_1	22.64	
P_9L_1	22.64	
P_1L_2	8.35	
P_2L_2	287.13	
		3564.96
P_1L_3	8.16	
P_2L_3	1645.95	
P_3L_3	649.24	
P_4L_3	280.51	
P_5L_3	981.10	

P_1L_4	8.33	
P_2L_4	1001.42	
P_3L_4	286.32	1602.72
P_4L_4	5.08	
P_5L_4	5.08	
P_6L_4	5.08	
P_7L_4	5.08	
P_8L_4	286.32	
P_1L_5	22.56	
P_2L_5	22.56	
P_3L_5	22.56	
P_4L_5	22.56	175.90
P_5L_5	22.56	
P_6L_5	9.00	
P_7L_5	9.00	
P_8L_5	22.56	
P_9L_5	22.56	

Table 3.2 : Distribution of Overturning Moment to Piers in short wall, as Axial Forces:

Pier No.	x_i (m)	A_i (m^2)	A_i X x_i (m^3)	$y_{NA} = \sum(A_i*x_i)/\sum A_i$ (m)	Distance of the central line of pier from N.A.
P_1S_1	3.45	2.76	9.52		4.05
P_2S_1	11.55	2.76	31.88		-4.05
		5.52	**41.40**	**7.50**	
P_1S_2	3.75	3.00	11.25		3.70
P_2S_2	11.85	2.52	29.86		-4.40
		5.52	**41.11**	**7.45**	
P_1S_3	2.33	1.86	4.32		5.06
P_2S_3	6.68	0.66	4.41		0.71
P_3S_3	7.73	0.18	1.39		-0.34
P_4S_3	12.68	1.86	23.58		-5.29
		4.56	**33.70**	**7.39**	
P_1S_4	2.33	1.86	4.32		0.33
P_2S_4	6.08	0.18	1.09		-3.42
		2.04	**5.42**	**2.66**	
P_1S_5	3.15	2.52	7.94		0.00
		2.52	**7.94**	**3.15**	
P_1S_6	3.15	2.52	7.94		4.35
P_2S_6	11.85	2.52	29.86		-4.35
		5.04	**37.80**	**7.50**	
P_1S_7	3.15	2.52	7.94		4.35
P_2S_7	11.85	2.52	29.86		-4.35
		5.04	**37.80**	**7.50**	
P_1S_8	0.50	0.40	0.20		7.00
P_2S_8	6.09	0.18	1.10		1.42
P_3S_8	8.93	0.18	1.61		-1.42
P_4S_8	14.50	0.40	5.80		-7.00
		1.16	**8.70**	**7.50**	

Table 3.3 : Distribution of Overturning Moment to Piers in long wall, as Axial Forces:

Pier No.	y_i (m)	A_i (m^2)	A_i X y_i(m^3)	$y_{NA} = \sum(A_i*y_i)/\sum A_i$ (m)	Distance of the central line of pier from N.A.
P_1L_1	0.20	0.16	0.032		12.40
P_2L_1	3.30	0.16	0.528		9.30
P_3L_1	6.40	0.16	1.024		6.20
P_4L_1	9.50	0.16	1.52		3.10
P_5L_1	12.60	0.16	2.016		0.00
P_6L_1	15.70	0.16	2.512		-3.10
P_7L_1	18.80	0.16	3.008		-6.20
P_8L_1	21.90	0.16	3.504		-9.30
P_9L_1	25.00	0.16	4		-12.40
$\sum$		**1.44**	**18.144**	**12.60**	
P_1L_2	0.20	0.08	0.016		1.94
P_2L_2	2.55	0.38	0.969		-0.41
$\sum$		**0.46**	**0.985**	**2.14**	
P_1L_3	0.20	0.16	0.032		12.67
P_2L_3	5.65	3.24	18.306		7.22
P_3L_3	14.15	1.40	19.81		-1.28
P_4L_3	18.05	0.76	13.718		-5.18
P_5L_3	22.70	2.00	45.4		-9.83
$\sum$		**7.56**	**97.266**	**12.87**	
P_1L_4	0.20	0.16	0.032		10.07
P_2L_4	4.10	2.00	8.2		6.17
P_3L_4	8.75	0.76	6.65		1.52
P_4L_4	12.60	0.16	2.016		-2.33
P_5L_4	15.70	0.16	2.512		-5.43
P_6L_4	18.80	0.16	3.008		-8.53
P_7L_4	21.90	0.16	3.504		-11.63
P_8L_4	24.25	0.76	18.43		-13.98
$\sum$		**4.32**	**44.352**	**10.27**	
P_1L_5	0.20	0.16	0.032		12.40
P_2L_5	3.30	0.16	0.528		9.30
P_3L_5	6.40	0.16	1.024		6.20
P_4L_5	9.50	0.16	1.52		3.10
P_5L_5	12.60	0.16	2.016		0.00

P_6L_5	15.70	0.16	2.512		-3.10
P_7L_5	18.80	0.16	3.008		-6.20
P_8L_5	21.90	0.16	3.504		-9.30
P_9L_5	25.00	0.16	4		-12.40
$\sum$		**1.44**	**18.144**	**12.60**	

Lateral Force Distribution among short walls:

Wall Name	Floor Level	Dist. Factor	Lateral Force	Total Overturning Moment (M_o)
S_1	Roof	0.28	519.06	
	Second	0.47	869.40	17255.89
	First	0.21	384.92	
	GF	0.04	71.82	
S_2	Roof	0.28	321.34	
	Second	0.47	538.22	10682.64
	First	0.21	238.29	
	GF	0.04	44.46	
S_3	Roof	0.28	275.82	
	Second	0.47	461.99	9169.56
	First	0.21	204.54	
	GF	0.04	38.16	
S_4	Roof	0.28	103.89	
	Second	0.47	174.02	3453.87
	First	0.21	77.04	
	GF	0.04	14.37	
S_5	Roof	0.28	99.64	
	Second	0.47	166.89	3312.47

	First	0.21	73.89	
	GF	0.04	13.79	
S_6	Roof	0.28	205.31	
	Second	0.47	343.88	6825.33
	First	0.21	152.25	
	GF	0.04	28.41	
S_7	Roof	0.28	211.34	
	Second	0.47	353.98	7025.73
	First	0.21	156.72	
	GF	0.04	29.24	
S_8	Roof	0.28	16.85	
	Second	0.47	28.22	560.18
	First	0.21	12.50	
	GF	0.04	2.33	

Lateral Force Distribution among long walls:

Wall Name	Floor Level	Dist. Factor	Lateral Force	Total Overturning Moment (M_o)
L_1	Roof	0.28	57.31	
	Second	0.47	95.99	1905.25
	First	0.21	42.50	
	GF	0.04	7.93	
L_2	Roof	0.28	83.12	
	Second	0.47	139.22	2763.29
	First	0.21	61.64	

	GF	0.04	11.50	
L_3	Roof	0.28	1002.83	
	Second	0.47	1679.70	33338.62
	First	0.21	743.67	
	GF	0.04	138.75	
L_4	Roof	0.28	450.85	
	Second	0.47	755.15	14988.26
	First	0.21	334.34	
	GF	0.04	62.38	
L_5	Roof	0.28	49.48	
	Second	0.47	82.88	1644.94
	First	0.21	36.69	
	GF	0.04	6.85	

Table 4 : Distribution of Overturning Moment to Piers as Axial Forces:

Pier No.	A_i (m^2)	$(y_b)_i$ (m)	$A_i(y_b)_i^2$ (m^4)	$(I_g)_i$ (m^4)	$(I_{NA})_i$ (m^4)	$A_i(y_b)_i$ (m^3)	$A_i(y_b)_i/\sum(I_{NA})$ (m^{-1})	$(P_e)_i$ (KN)
P_1S_1	2.760	4.050	45.271	10.950	56.221	11.178	0.099	1715.423
P_2S_1	2.760	-4.050	45.271	10.950	56.221	-11.178	-0.099	-1715.423
$\sum$	**5.520**		**90.542**	**21.901**	**112.442**	**0.000**	**0.000**	**0.000**
P_1S_2	3.000	3.698	41.022	14.063	55.084	11.093	0.099	1055.704
P_2S_2	2.520	-4.402	48.835	8.335	57.170	-11.093	-0.099	-1055.704
$\sum$	**5.520**		**89.857**	**22.397**	**112.255**	**0.000**	**0.000**	**0.000**

P_1S_3	1.860	5.064	47.707	3.351	51.058	9.420	0.088	808.146
P_2S_3	0.660	0.714	0.337	0.150	0.487	0.472	0.004	40.455
P_3S_3	0.180	-0.336	0.020	0.003	0.023	-0.060	-0.001	-5.181
P_4S_3	1.860	-5.286	51.962	3.351	55.314	-9.831	-0.092	-843.420
∑	**4.560**		**100.027**	**6.856**	**106.882**	**0.000**	**0.000**	**0.000**
P_1S_4	1.860	0.331	0.204	3.351	3.555	0.615	0.109	375.397
P_2S_4	0.180	-3.419	2.104	0.003	2.107	-0.615	-0.109	-375.397
∑	**2.040**		**2.308**	**3.355**	**5.662**	**0.000**	**0.000**	**0.000**
P_1S_5	2.520	0.000	0.000	8.335	8.335	0.000	0.000	0.000
∑	**2.520**		**0.000**	**8.335**	**8.335**	**0.000**	**0.000**	**0.000**
P_1S_6	2.520	4.350	47.685	8.335	56.020	10.962	0.098	667.796
P_2S_6	2.520	-4.350	47.685	8.335	56.020	-10.962	-0.098	-667.796
∑	**5.040**		**95.369**	**16.670**	**112.039**	**0.000**	**0.000**	**0.000**
P_1S_7	2.520	4.350	47.685	8.335	56.020	10.962	0.098	687.403
P_2S_7	2.520	-4.350	47.685	8.335	56.020	-10.962	-0.098	-687.403
∑	**5.040**		**95.369**	**16.670**	**112.039**	**0.000**	**0.000**	**0.000**
P_1S_8	0.40	7.002	19.609	0.033	19.642	2.801	0.070	39.223
P_2S_8	0.18	1.417	0.361	0.003	0.364	0.255	0.006	3.571
P_3S_8	0.18	-1.423	0.365	0.003	0.368	-0.256	-0.006	-3.588
P_4S_8	0.40	-6.998	19.591	0.033	19.625	-2.799	-0.070	-39.205
∑	**1.16**		**39.926**	**0.073**	**39.999**	**0.000**	**0.000**	**0.000**
P_1L_1	0.160	12.400	24.602	0.002	24.604	1.984	0.022	40.964
P_2L_1	0.160	9.300	13.838	0.002	13.841	1.488	0.016	30.723

P_3L_1	0.160	6.200	6.150	0.002	6.153	0.992	0.011	20.482
P_4L_1	0.160	3.100	1.538	0.002	1.540	0.496	0.005	10.241
P_5L_1	0.160	0.000	0.000	0.002	0.002	0.000	0.000	0.000
P_6L_1	0.160	-3.100	1.538	0.002	1.540	-0.496	-0.005	-10.241
P_7L_1	0.160	-6.200	6.150	0.002	6.153	-0.992	-0.011	-20.482
P_8L_1	0.160	-9.300	13.838	0.002	13.841	-1.488	-0.016	-30.723
P_9L_1	0.160	-12.400	24.602	0.002	24.604	-1.984	-0.022	-40.964
$\sum$	**1.440**		**92.256**	**0.019**	**92.275**	**0.000**	**0.000**	**0.000**
P_1L_2	0.400	1.941	1.507	0.001	1.509	0.777	0.400	1105.941
P_2L_2	1.900	-0.409	0.317	0.114	0.432	-0.777	-0.400	-1105.941
$\sum$	**2.300**		**1.825**	**0.115**	**1.940**	**0.000**	**0.000**	**0.000**
P_1L_3	0.160	12.666	25.668	0.002	25.670	2.027	0.005	155.649
P_2L_3	3.240	7.216	168.703	17.715	186.418	23.379	0.054	1795.667
P_3L_3	1.400	-1.284	2.309	1.429	3.738	-1.798	-0.004	-138.079
P_4L_3	0.760	-5.184	20.425	0.229	20.654	-3.940	-0.009	-302.609
P_5L_3	2.000	-9.834	193.420	4.167	197.587	-19.668	-0.045	-1510.629
$\sum$	**7.560**		**410.525**	**23.541**	**434.066**	**0.000**	**0.000**	**0.000**
P_1L_4	0.160	10.067	16.214	0.002	16.216	1.611	0.006	84.363
P_2L_4	2.000	6.167	76.056	4.167	80.222	12.333	0.043	645.993
P_3L_4	0.760	1.517	1.748	0.229	1.977	1.153	0.004	60.374
P_4L_4	0.160	-2.333	0.871	0.002	0.873	-0.373	-0.001	-19.554
P_5L_4	0.160	-5.433	4.723	0.002	4.726	-0.869	-0.003	-45.534
P_6L_4	0.160	-8.533	11.651	0.002	11.653	-1.365	-0.005	-71.513
P_7L_4	0.160	-11.633	21.654	0.002	21.656	-1.861	-0.007	-97.493
P_8L_4	0.760	-13.983	148.606	0.229	148.834	-10.627	-0.037	-556.636

$\sum$	**4.320**		**281.522**	**4.635**	**286.157**	**0.000**	**0.000**	**0.000**
P_1L_5	0.160	12.400	24.602	0.002	24.604	1.984	0.022	35.368
P_2L_5	0.160	9.300	13.838	0.002	13.841	1.488	0.016	26.526
P_3L_5	0.160	6.200	6.150	0.002	6.153	0.992	0.011	17.684
P_4L_5	0.160	3.100	1.538	0.002	1.540	0.496	0.005	8.842
P_5L_5	0.160	0.000	0.000	0.002	0.002	0.000	0.000	0.000
P_6L_5	0.160	-3.100	1.538	0.002	1.540	-0.496	-0.005	-8.842
P_7L_5	0.160	-6.200	6.150	0.002	6.153	-0.992	-0.011	-17.684
P_8L_5	0.160	-9.300	13.838	0.002	13.841	-1.488	-0.016	-26.526
P_9L_5	0.160	-12.400	24.602	0.002	24.604	-1.984	-0.022	-35.368
$\sum$	**1.440**		**92.256**	**0.019**	**92.275**	**0.000**	**0.000**	**0.000**

Load Calculation on Walls:

Wall Name	Dead Load	Live Load
S_1	423.60	47.04
S_2	785.60	174.48
S_3	724.00	160.80
S_4	724.00	160.80
S_5	724.00	160.80
S_6	724.00	160.80
S_7	724.00	160.80
S_8	362.00	80.40
L_1	799.28	177.52
L_2	81.05	18.00
L_3	1888.53	419.44
L_4	1888.53	419.44
L_5	799.28	177.52

Table 3.4 : Forces in different piers due to different loads:

Pier No.	$(P_d)_i$ (KN)	$(P_l)_i$ (KN)	$(P_e)_i$ (KN)	$(V_e)_i$ (KN)
P_1S_1	211.80	23.52	1429.52	768.83
P_2S_1	211.80	23.52	-1429.52	768.83
$\sum$	**423.60**	**47.04**	**0.00**	**1537.67**
P_1S_2	424.22	94.22	879.75	519.85
P_2S_2	361.37	80.26	-879.75	432.08
$\sum$	**785.60**	**174.48**	**0.00**	**951.93**
P_1S_3	253.40	56.28	673.46	300.19
P_2S_3	137.56	30.55	33.71	73.87
P_3S_3	79.64	17.69	-4.32	9.23
P_4S_3	253.40	11.26	-702.85	433.80
$\sum$	**724.00**	**115.78**	**0.00**	**817.10**
P_1S_4	613.07	136.16	312.83	303.97
P_2S_4	122.61	27.23	-312.83	3.80
$\sum$	**735.68**	**163.39**	**0.00**	**307.77**
P_1S_5	724.00	160.80	0.00	295.17
$\sum$	**724.00**	**160.80**	**0.00**	**295.17**
P_1S_6	362.00	80.40	556.50	304.10
P_2S_6	362.00	80.40	-556.50	304.10
$\sum$	**724.00**	**160.80**	**0.00**	**608.20**
P_1S_7	362.00	80.40	572.84	313.03
P_2S_7	362.00	80.40	-572.84	313.03

∑	**724.00**	**160.80**	**0.00**	**626.06**
P_1S_8	82.66	18.36	32.69	22.41
P_2S_8	98.34	21.84	2.98	2.55
P_3S_8	98.34	21.84	-2.99	2.55
P_4S_8	82.66	18.36	-32.67	22.41
∑	**362.00**	**80.40**	**0.00**	**49.92**
P_1L_1	55.51	12.33	34.14	18.86
P_2L_1	98.32	21.84	25.60	18.86
P_3L_1	98.32	21.84	17.07	18.86
P_4L_1	98.32	21.84	8.53	18.86
P_5L_1	98.32	21.84	0.00	18.86
P_6L_1	98.32	21.84	-8.53	18.86
P_7L_1	98.32	21.84	-17.07	18.86
P_8L_1	98.32	21.84	-25.60	18.86
P_9L_1	55.51	12.33	-34.14	18.86
∑	**799.28**	**177.52**	**0.00**	**169.78**
P_1L_2	23.16	5.14	921.62	6.96
P_2L_2	57.89	12.86	-921.62	239.28
∑	**81.05**	**18.00**	**0.00**	**246.24**
P_1L_3	74.94	16.64	129.71	6.80
P_2L_3	753.16	167.28	1496.39	1371.63
P_3L_3	408.43	90.71	-115.07	541.03
P_4L_3	232.32	51.60	-252.17	233.76
P_5L_3	419.67	93.21	-1258.86	817.58
∑	**1888.53**	**419.44**	**0.00**	**2970.80**
P_1L_4	74.94	16.64	70.30	6.94
P_2L_4	464.64	103.20	538.33	834.52

P_3L_4	288.53	64.08	50.31	238.60
P_4L_4	232.32	51.60	-16.30	4.23
P_5L_4	232.32	51.60	-37.94	4.23
P_6L_4	232.32	51.60	-59.59	4.23
P_7L_4	176.11	39.11	-81.24	4.23
P_8L_4	187.35	41.61	-463.86	238.60
$\sum$	**1888.53**	**419.44**	**0.00**	**1335.60**
P_1L_5	55.51	12.33	29.47	18.80
P_2L_5	98.32	21.84	22.10	18.80
P_3L_5	98.32	21.84	14.74	18.80
P_4L_5	98.32	21.84	7.37	18.80
P_5L_5	98.32	21.84	0.00	18.80
P_6L_5	98.32	21.84	-7.37	7.50
P_7L_5	98.32	21.84	-14.74	7.50
P_8L_5	98.32	21.84	-22.10	18.80
P_9L_5	55.51	12.33	-29.47	18.80
$\sum$	**799.28**	**177.52**	**0.00**	**146.58**

Table 3.5 : As per IS 1905:1987

Permissible Compressive Stress = 1.9 N/mm^2

Permissible Bending Stress = 2.375 N/mm^2

Pier Nos.	Direct Stress (N/mm^2)	Overturning Stress (N/mm^2)	Bending Stress (N/mm^2)	(fa/Fa) + (fb/Fb) $\leq$ 1.33
P_1S_1	0.085	0.622	0.218	0.46
P_2S_1	0.085	-0.622	0.218	0.46
P_1S_2	0.173	0.352	0.175	0.35
P_2S_2	0.175	-0.419	0.206	0.40
P_1S_3	0.166	0.434	0.262	0.43
P_2S_3	0.255	0.061	0.513	0.38
P_3S_3	0.541	-0.029	0.616	0.56
P_4S_3	0.142	-0.453	0.271	0.43
P_1S_4	0.403	0.202	0.266	0.43
P_2S_4	0.832	-2.086	0.355	**1.69**
P_1S_5	0.351	0.000	0.201	0.27
P_1S_6	0.176	0.265	0.207	0.32
P_2S_6	0.176	-0.265	0.207	0.32
P_1S_7	0.176	0.273	0.213	0.33
P_2S_7	0.176	-0.273	0.213	0.33
P_1S_8	0.253	0.098	0.605	0.44

P_2S_8	0.668	0.020	0.340	0.51
P_3S_8	0.668	-0.020	0.340	0.51
P_4S_8	0.253	-0.098	0.605	0.44
P_1L_1	0.424	0.256	1.592	1.03
P_2L_1	0.751	0.192	1.592	1.17
P_3L_1	0.751	0.128	1.592	1.13
P_4L_1	0.751	0.064	1.592	1.10
P_5L_1	0.751	0.000	1.592	1.07
P_6L_1	0.751	-0.064	1.592	1.10
P_7L_1	0.751	-0.128	1.592	1.13
P_8L_1	0.751	-0.192	1.592	1.17
P_9L_1	0.424	-0.256	1.592	1.03
P_1L_2	0.354	2.765	1.644	**2.33**
P_2L_2	0.186	-0.582	2.505	**1.46**
P_1L_3	0.572	0.973	0.803	1.15
P_2L_3	0.284	0.554	0.395	0.61
P_3L_3	0.357	-0.099	0.835	0.59
P_4L_3	0.374	-0.398	1.224	0.92
P_5L_3	0.256	-0.755	0.618	0.79
P_1L_4	0.572	0.527	0.820	0.92
P_2L_4	0.284	0.323	0.631	0.59
P_3L_4	0.464	0.079	1.249	0.81
P_4L_4	1.774	-0.122	0.596	1.25
P_5L_4	1.774	-0.285	0.596	1.33
P_6L_4	1.774	-0.447	0.596	**1.42**
P_7L_4	1.345	-0.609	0.596	1.28
P_8L_4	0.301	-0.732	1.249	1.07
P_1L_5	0.424	0.221	1.586	1.01

P_2L_5	0.751	0.166	1.586	1.15
P_3L_5	0.751	0.111	1.586	1.12
P_4L_5	0.751	0.055	1.586	1.09
P_5L_5	0.751	0.000	1.586	1.06
P_6L_5	0.751	-0.055	0.885	0.80
P_7L_5	0.751	-0.111	0.885	0.83
P_8L_5	0.751	-0.166	1.586	1.15
P_9L_5	0.424	-0.221	1.586	1.01

*In case of P_2S_4, P_1L_2, P_2L_2, P_6L_4 extra steel will be provided to make it safe

Table 3.6 : Steel provided in form of Bars and Flats:

Pier No.	$(M_e)_i = (V_e)_i.(h_i/2)$ (KNm)	Effective Depth (mm)	Area of Jamb Steel (A_s) (mm^2)	No. of Bars	Steel provided (In form of flat on both faces) (t mm X b mm)
P_1S_1	691.95	6860	487.28	1 @ 25Φ	
P_2S_1	691.95	6860	438.55	1 @ 25Φ	
P_1S_2	655.01	7460	381.75	-	8 X 125
P_2S_2	544.42	6260	378.12	-	8 X 125
P_1S_3	378.24	4610	356.73	-	8 X 125
P_2S_3	93.08	1610	251.37	-	8 X 125
P_3S_3	8.31	410	88.13	-	8 X 125
P_4S_3	390.42	4610	368.21	-	8 X 125
P_1S_4	383.00	4610	361.22	-	8 X 125
P_2S_4	4.79	410	50.82	-	8 X 125
P_1S_5	531.31	6260	369.02	-	8 X 125
P_1S_6	547.38	6260	380.18	-	8 X 125
P_2S_6	547.38	6260	380.18	-	8 X 125
P_1S_7	563.46	6260	391.34	-	8 X 125
P_2S_7	563.46	6260	391.34	-	8 X 125
P_1S_8	40.33	960	182.68	-	8 X 125
P_2S_8	4.59	410	48.68	-	8 X 125

P_3S_8	4.59	410	48.68	-	8 X 125
P_4S_8	40.33	960	182.68	-	8 X 125
P_1L_1	16.98	360	205.04	1 @ 25Φ	-
P_2L_1	16.98	360	205.04	1 @ 25Φ	-
P_3L_1	16.98	360	205.04	1 @ 25Φ	-
P_4L_1	16.98	360	205.04	1 @ 25Φ	-
P_5L_1	16.98	360	205.04	1 @ 25Φ	-
P_6L_1	16.98	360	205.04	1 @ 25Φ	-
P_7L_1	16.98	360	205.04	1 @ 25Φ	-
P_8L_1	16.98	360	205.04	1 @ 25Φ	-
P_9L_1	16.98	360	205.04	1 @ 25Φ	-
P_1L_2	8.77	360	105.89	-	8 X 125
P_2L_2	301.49	1860	704.75	-	8 X 125
P_1L_3	8.57	360	103.44	-	8 X 125
P_2L_3	1728.25	8060	932.27	-	8 X 125
P_3L_3	681.70	3460	856.62	-	8 X 125
P_4L_3	294.54	1860	688.49	-	8 X 125
P_5L_3	1030.15	4960	903.01	-	8 X 125
P_1L_4	8.74	360	105.59	-	8 X 125
P_2L_4	1051.49	4960	921.71	-	8 X 125
P_3L_4	300.64	1860	702.76	-	8 X 125
P_4L_4	6.35	360	76.72	-	8 X 125
P_5L_4	6.35	360	76.72	-	8 X 125
P_6L_4	6.35	360	76.72	-	8 X 125
P_7L_4	6.35	360	76.72	-	8 X 125
P_8L_4	300.64	1860	702.76	-	8 X 125
P_1L_5	16.92	360	204.33	1 @ 25Φ	-

P_2L_5	16.92	360	204.33	1 @ 25Φ	-
P_3L_5	16.92	360	204.33	1 @ 25Φ	-
P_4L_5	16.92	360	204.33	1 @ 25Φ	-
P_5L_5	16.92	360	204.33	1 @ 25Φ	-
P_6L_5	9.45	360	114.07	1 @ 25Φ	-
P_7L_5	9.45	360	114.07	1 @ 25Φ	-
P_8L_5	16.92	360	204.33	1 @ 25Φ	-
P_9L_5	16.92	360	204.33	1 @ 25Φ	-

SEQUENCE OF EXECUTION OF RECOMMENDED STRUCTURAL RETROFITTING MEASURES

1.) At plinth level coring of 50 mm dia. maintaining the center of core hole at 35 mm from face of wall. Similar coring is done at other face as well. Thereafter coring of 32 mm dia. at 300 mm c/c distance should be done in transverse direction, such that the centre of the transverse core hole is 40mm above the centre of the longitudinal core hole. Insert bars of 25 mm in 50 mm dia. core hole with chairs of 12 mm placed beneath the 25 mm bar at every 600 mm c/c. The 8 mm dia. bars is then inserted in 32 mm dia. core hole. tying of transverse bars with longitudinal bars is done by binding wires to form a ladder shaped steel band.

2.) Same procedure mentioned above is to be followed for lintel band.

3.) At sill level coring of 32 mm dia. is done through the junction of long wall with cross walls such that the core hole center is 51 mm from face of cross wall and extends 1 m beyond the thickness of long wall this is to be done for both face of cross wall. Once coring is done 12 mm dia. bar is then inserted to these core holes and binding of these bars with the vertical bars is done to make proper stitching together of walls.

4.) Coring of 50 mm dia. for providing vertical reinforcement is then done from roof level to bottom of foundation within the wall at distance of 200 mm from face of the long wall along both face of cross wall.

5.) The same method as mentioned in step 4 above is to be followed for providing the jamb steel for all piers in exterior walls.

6.) Once all coring is done and respective bars are inserted, micro concreting of grade M30 is carried out to fill the vacant core holes area. This results in making a proper skeleton system within the structure with minimum activities of the hospital being affected.

7.) For the piers in interior walls flats of size 125 mm width of 8 mm thickness is to be provided as described in the following steps.

8.) The room in which the jamb steel around openings is to be provided should be vacated. Thereafter chiseling of the wall for a width of 150 mm is done on both the vertical sides of the opening on both face of the wall. The specified flats are placed on both faces of the walls such that one end along its width is flush with the opening and joined together by nut bolts of 8 mm dia. rebar in drilled hole @ 300 mm c/c.

9.) At lintel level the transverse rebar (8 mm dia.) is bolted to both plates to make proper connections. In case of door openings apart from the connections at lintel and sill level, the flats placed vertically are extended beyond the floor finish upto the concrete slab.

10.) During performing step 7 above simultaneously grouting in walls and pilaster construction (if required) is to be done.

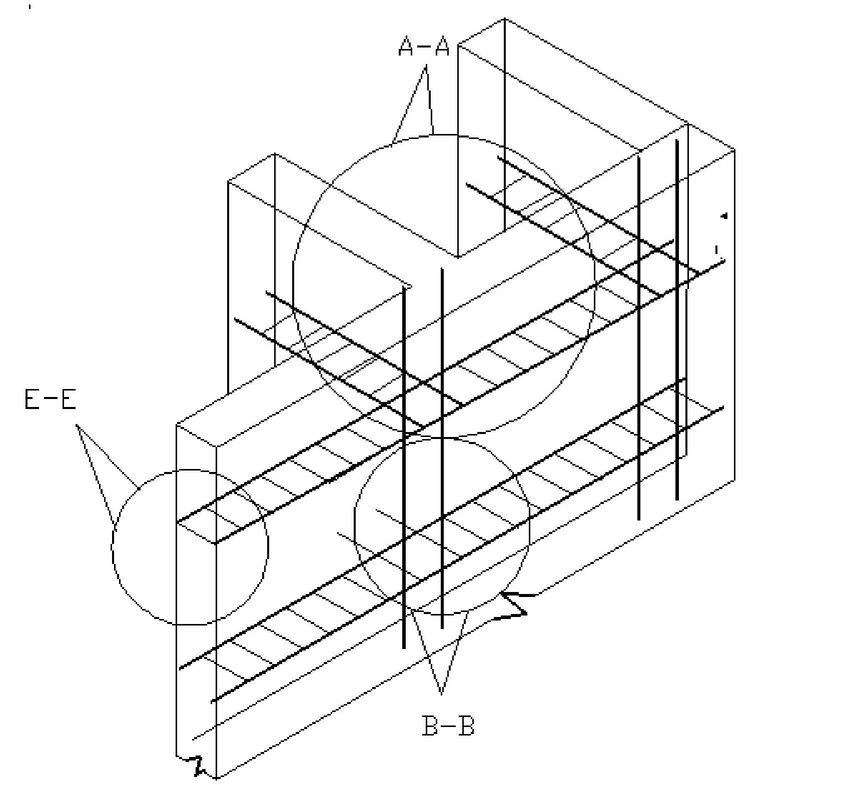
11.) False ceiling and entire roofing material is removed for providing roof band and gable band.

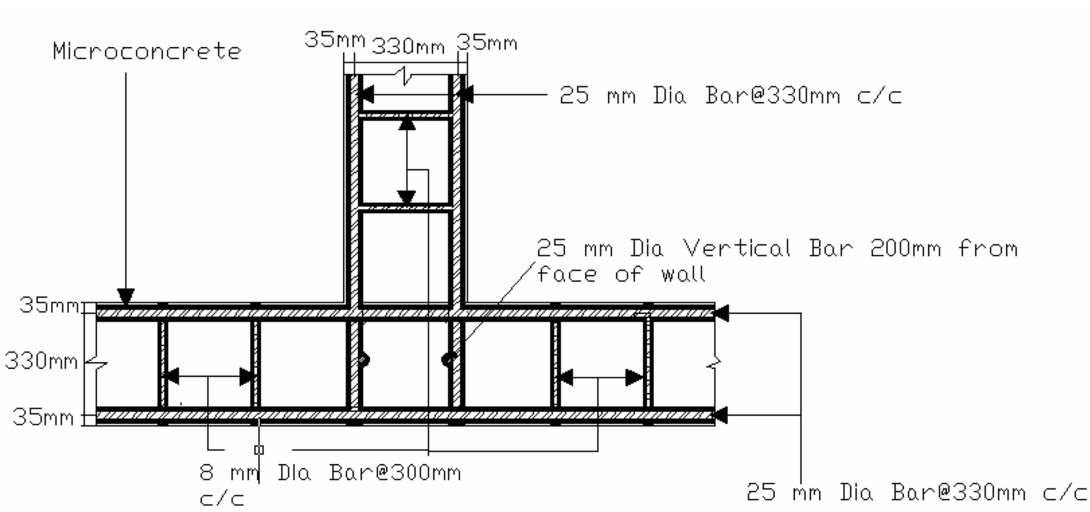
12.) Roof band and gable band is provided as `I` section of size 75 x 40 mm and after that trusses are placed on the roof beam. The rafters are then attached with the roof and purlins with the gable band by 12 mm dia. rod in the form of U hook which is welded with the `I` section.

13.) In-Plane bracing at tie level is done with 25 mm dia. bars which are fixed the wooden tie member by U hook of 12 mm dia. bars.

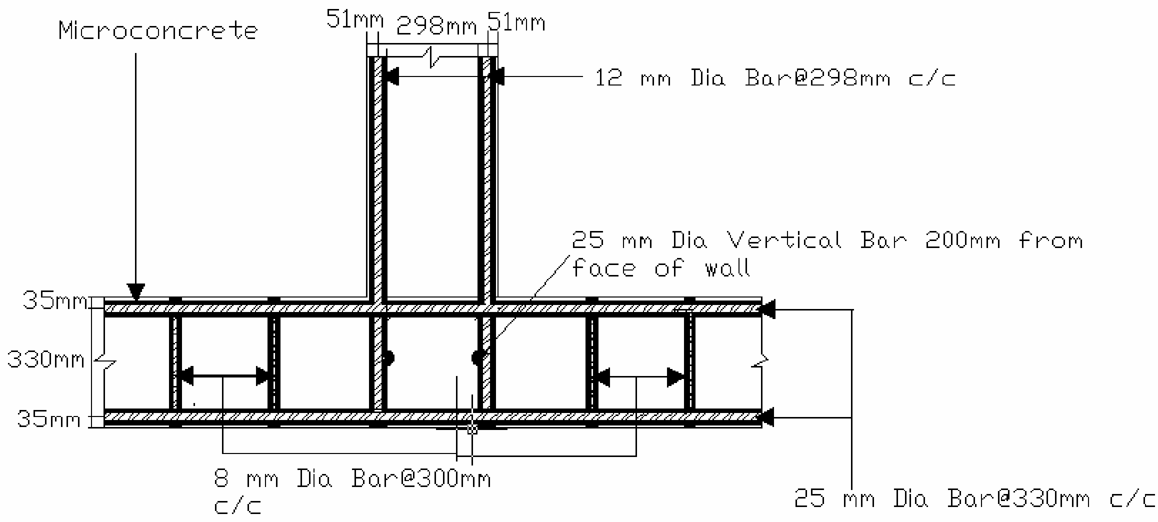
14.) Once the above bands are constructed the false ceiling is then hung from the purlins with secure connection to prevent any sway.

Fig. 3.3 : Arrangement of bands is shown in figure-:

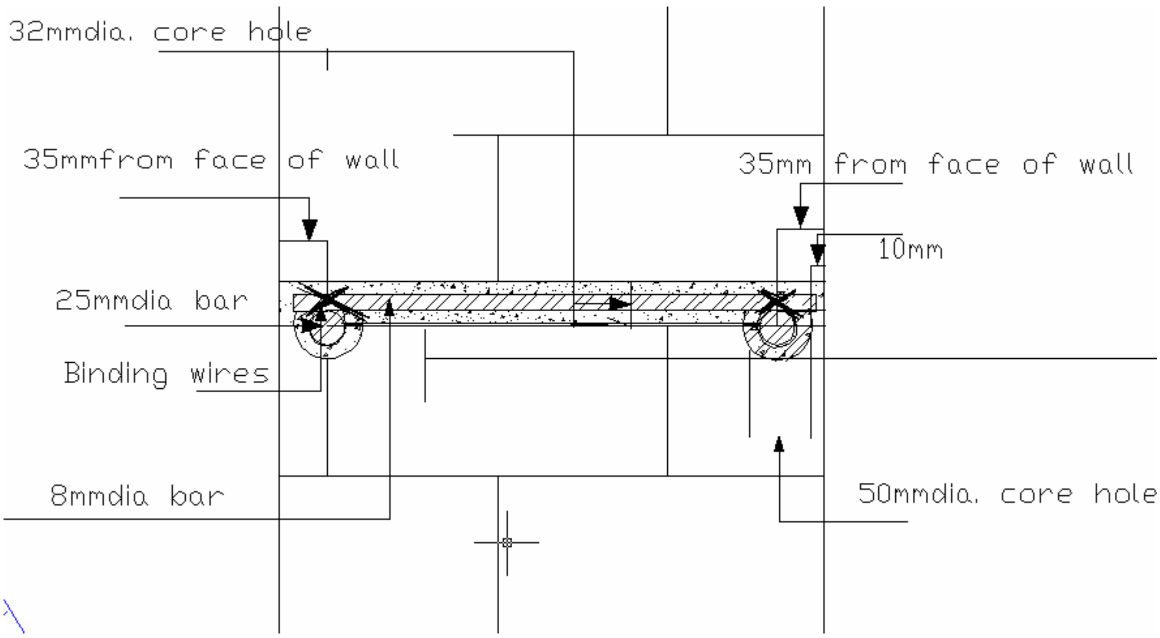




A — A

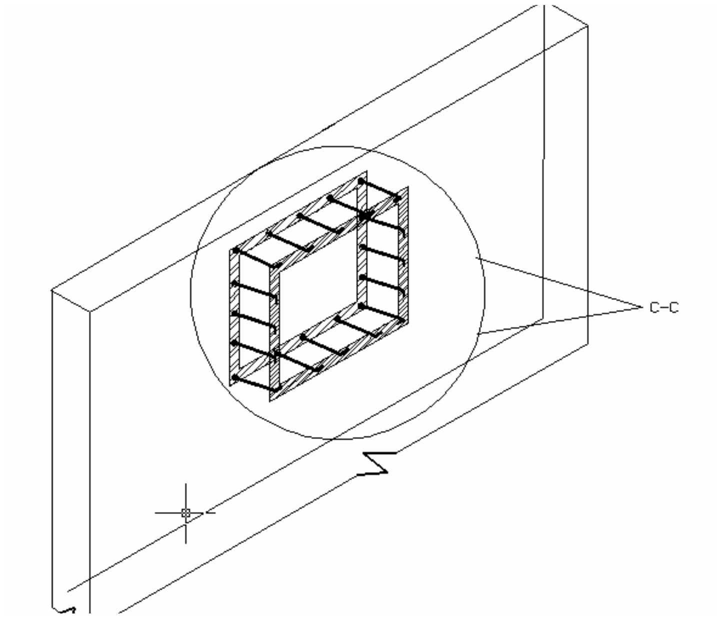

Microconcrete
51mm
298mm
51mm
12 mm Dia Bar@298mm c/c
25 mm Dia Vertical Bar 200mm from face of wall
35mm
330mm
35mm
8 mm Dia Bar@300mm c/c
25 mm Dia Bar@330mm c/c

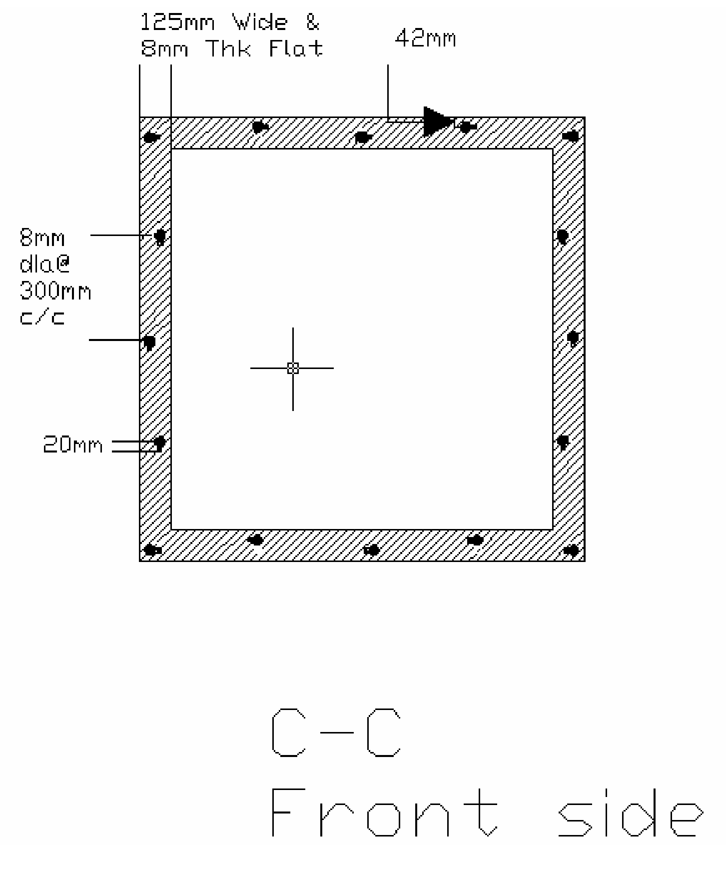
B — B


32mmdia. core hole
35mmfrom face of wall
35mm from face of wall
10mm
25mmdia bar
Binding wires
8mmdia bar
50mmdia. core hole

E - E

Fig. 3.4 : Arrangement of Flats at window openings-:





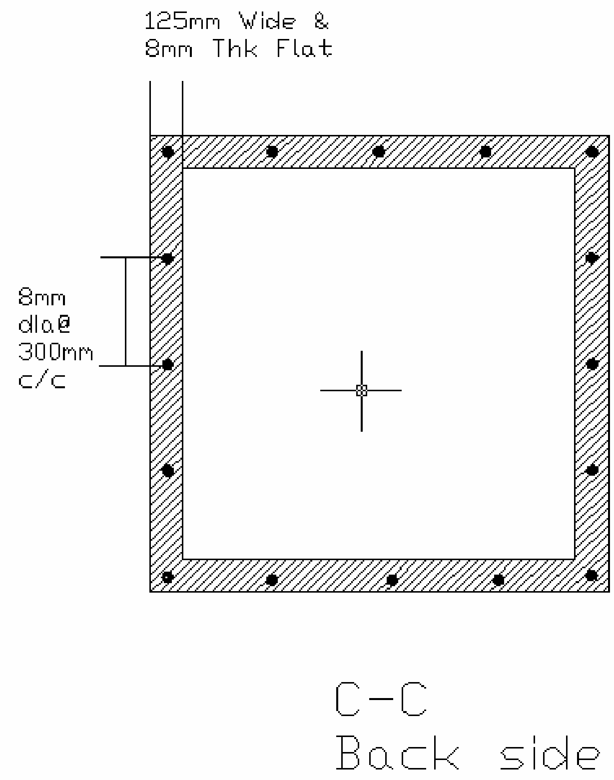
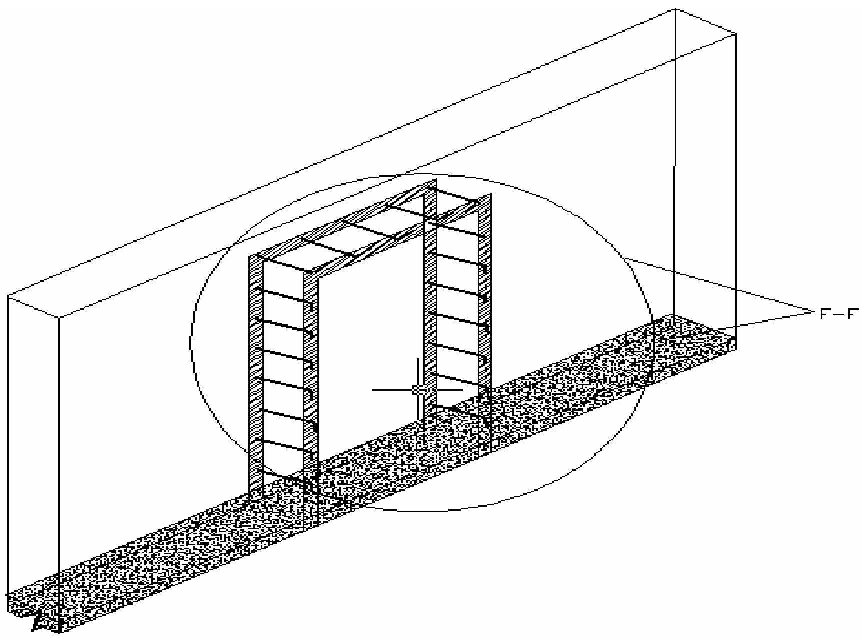
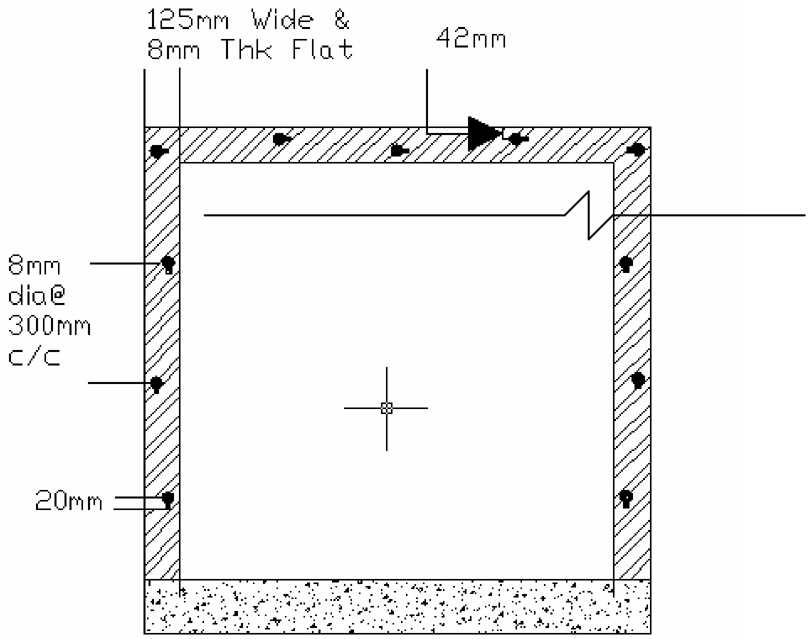
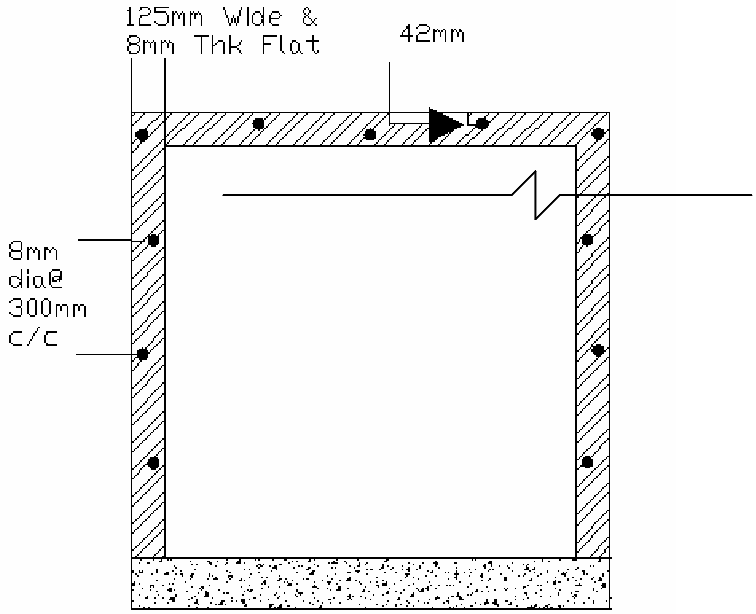


Fig. 3.5 : Arrangement of Flats at Door openings-:





F-F
Front Side



F-F
Back Side

3.7 COST CALCULATION & COMPARISON WITH RATE OF NEW CONSTRUCTION

(1A) Cost of structural retrofitting

S.No.	Description	Unit	Qty.	Rate(Rs.)	Amount(Rs.)
A	Steel bars to be provided upto 8mm dia. including fabrication charges	kg	439.47	44.87	**19719.02**
B	Steel bars of 25 mm to be provided including fabrication charges	kg	11592.49	44.34	**514011.00**
C	Steel bars of 12 mm to be provided including fabrication charges	kg	83.2	44.87	**3733.19**
D	Providing and fixing of flats upto size of 10mm thick.	kg	2468.88	42.77	**105593.99**
E	providing and fixing of I section of 75X40	kg	1093.5	42.15	**45091.03**
F	Quantity of coring of following mm dia. to be done including all charges I) 32 mm = 1363.16 meters II) 50 mm = 2703.71 meters	m	4066.87	1270.39	**5166510.98**
G	Amount of micro concreting to be done including all the charges	cum	2.095	55556.8	**116376.88**
H	Amount of grouting of cement to be done	cum	105.56	11440	**1207592.1**
I	Demolishing cement concrete work manually/ by mechanical means including stacking of serviceable material and disposal of unserviceable material within 50 meters leas as per direction of Engineer-in-charge. In cement mortar.	cum	1.52	226.90	**344.88**
J	Providing and fixing bolts of 8 mm dia. including nuts and washers complete.	kg	79.45	62.75	**4985.49**

K	Providing plaster with a mixture of cement & sand 1:4 (1 cement: 4 fine sand) of thickness 15 mm.	sqm	156.84	110	**56775.785**
L	Interior Finishing-Painting Distempering with oil bound washable distemper of approved brand and manufacture to give an even shade New work (two or more coats) over and including priming coat with cement primer	sqm	2628.54	41.55	**109215.837**
M	Exterior Finishing-Painting Finishing walls with textured exterior paint of required shade Two or more coat applied @ 3.28 ltr/10 sqm over and including base coat of water proofing cement paint applied @ 2.20 kg/10 sqm	sqm	625.5	96.25	**60204.375**
N	Scaffolding for painting, coring & plastering, providing double scaffolding system (cup lock type) on the exterior side, up to seven storey height made with 40 mm dia. M.S tube 1.5 m center to center horizontal & vertical tubes joining with cup & lock system with M.S tubes, M.S tube challis, M.S clamps and M.S stair case system in the scaffolding for working platform etc and maintaining in a serviceable condition for the required duration as approved and removing it thereafter. The scaffolding system shall be stiffened with bracings, runners, connection with the building etc where ever required for inspection of works at required locations with essential safety features for the workmen etc. complete as per directions and approval for engineer-in-charge. The lavational area of the scaffolding shall be measured for payment purpose	sqm	801.6	82.7	**66292.32**

O	Repair and strengthening of damaged porches by RCC	cum	7.5	3257.45	**24430.88**
P	Dismantling and removing from site wooden false ceiling.	sqm	378	28	**10584**
Q	Internal Water Supply & Sanitary Installations @ 10% of new construction	sqm	383.85	1666.1	**639532.49**
R	External Service connection @ 5 % of new construction	sqm	383.85	833.05	**319766.25**
S	Internal Electric Installation @ 12.5 % of new construction	sqm	383.5	2082.63	**798688.61**
	Sub Total				**9269449**
	Contingencies @ 5%				**463472.45**
	TOTAL				**9732921.45**
	Present cost Index for Delhi is 19%. Therefore Unit rate of civil works for retrofitting (Per sqm) area is as on date = **9732921.45*19%**				**1849255.08**
	TOTAL COST i/c Delhi cost index				**11582176.53**
	Per sqm rate for civil works for retrofitting				**9052.12**

(1B) Miscellaneous Costs / Special Feature Costs

2.5% of new construction			Rs/sqm	542.73

1)

TOTAL COST OF RETROFITTING (1A+1B)	Rs/sqm	**9594.85**

1.) Cost for Non-Structural Components/Contents/Equipments/Elements

Cost for non-structural components @ 5% of new construction	sqm	1279.5	**1085.45**	**1388833.28**

2.) Inconvenience & Shifting Cost

This cost includes the following:				
i) Renting of another equivalent area premises				
ii) Shifting of facilities to new premises.				
iii) Breakage of medical and non-medical equipment during shifting and erecting.				
iv) Creation of similar facilities in new premises i/c civil works.				
v) Inconvenience caused to patients and working staff during shifting.				
vi) Re-Shifting of facilities to retrofitted premises.				
vii) Breakage of medical and non-medical equipment during re-shifting and re-erecting.				
viii) Re-Creation of facilities in retrofitted structure.				
ix) Inconvenience caused to patients and working staff during re-shifting.	sqm	1279.5	@20% of New construction 4341.8	**5555333.1**

3.) Abstract Cost Sheet

Items	Unit	Area (sqm)	Rate	Amount (Rs.)
Total Cost of retrofitting	Rs./sqm	5487.5	9594.85	52651739.38
Non-Structural cost	Rs./sqm	5487.5	1085.45	5956406.88
Inconvenience & Shifting Cost	Rs./sqm	5487.5	4341.8	23825627.50
GRAND TOTAL				**82433773.76**

4.) Rate of New Construction

ANNEXURE-A			
COST ESTIMATE FOR RETROFITTING OF HOSPITAL BUILDING AT DHARAMSALA			
Based on C.P.W.D Plinth Area Rates-01.10.2007			
PAR NO.	**DESCRIPTION**	**UNIT**	**RATE (Rs.)**
2.0	**LOAD BEARING CONSTRUCTION**		
2.1	Floor Height 3.35 Mtr		
2.1.4	Three Storeyed	Sqm	8250.00
2.5	**EXTRA FOR**		
2.5.3	Every 0.30 mt. deeper foundations over normal depth of 1.20 mt. (on G.F. area only)	Sqm	150.00
2.6.1	Resisting earthquake forces	Sqm	588.00
2.7	Stronger structural member to take heavy load above 500Kgs./sqm upto 1000 Kgs,/sqm	Sqm	850.00
2.8	Large modules over 35 Sqm	Sqm	990.00
	A	TOTAL	10828.00
2.9	**FIRE FIGHTING**		
2.9.2	With Sprinkler System	Sqm	450.00
2.10	**FIRE ALARM SYSTEM**		
2.10.2	Automatic Fire Alarm System	sqm	300.00
2.11	Operation Theatre (OPD)	sqm	1235.00
	B	TOTAL	1985.00
3.0	**SERVICES**		
3.1	Internal Water Supply & Sanitary Installations	Sqm	10.0%
3.2	External Service connection	Sqm	5.0%
3.3	Internal Electric Installation	Sqm	12.50%
3.6	**Extra for**		

3.6.1	Power Wiring and Plug	Sqm	4.0%
3.6.3	Lighting Conductor		
3.6.3.1	upto 4 storeyed building	sqm	0.50%
3.6.4	Telephone Conduits	Sqm	0.50%
3.6.6	Computor Conduting	Sqm	0.50%
3.6.7	Quality Assurance	Sqm	1.00%
	Extra for Higher Specifications	Sqm	5.00%
	C	TOTAL	39.0%
5.0	**Water Tank (Rcc only)**		
5.1	Overhead tank without independent staging	per Litre	9.00
	minimum water rquirment70000 Ltr per day, hence rate is on 1000sqm area basis D	TOTAL	630.00
	Other Work		
1	COST FOR **HVAC** WORKS	Sqm	500.00
2	COST FOR SPECIAL FINISHES FOR HOSPITAL INTERIOR WORKS	Sqm	800.00
	E	Total	1300.00
	TOTAL OF A To E	**F**	**16661.00**
	NOTE:-		
1	Present Cost Index for Delhi is 19%. Therefore Unit Rate (per Sq.M.) area is as on date = 16478 x 19%	**G**	3165.59
	Total of F to G		19826.59
2	Other Work (Based on Market Rate) **F**		1300.00
			21126.59
	Net Total	**SAY Rs.**	**21127.00**

Table 3.8

ANNEXURE-B			
Based on C.P.W.D Plinth Area Rates-01.10.2007			
PAR NO.	**DESCRIPTION OF ITEM**	**UNIT**	**RATE (Rs.)**
6.0	**DEVELOPMENT OF SITE**		
6.1	Levelling	Sqm	55.00
6.2	Internal roads & paths	Sqm	83.00
6.3	Sewer	Sqm	63.00
6.4	Filter Water Supply		
6.4.1	Distribution lines 100mm dia and below	Sqm	46.00
6.5	Strom Water Drains	Sqm	50.00
6.6	Horticulture Operations	Sqm	47.00
	A	TOTAL	344.00
6.7	**Street Lighting**		
6.7.1	With HPSV LAMPS	Sqm	95.00
6.7.4	Exit Sign Board i/c electric signage	Sqm	50.00
	B	TOTAL	145.00
	TOTAL OF A To B	**C**	**489.00**
	NOTE:-		
1	Present Cost Index for Delhi is 19%. Therefore Unit Rate (per Sq.M.) area is as on date = 23396 x 19%	D	92.91
	Total of C to D		581.91
	Net Total	**SAY Rs.**	**582.00**
	TOTAL COST ANNEXURE (A +B)	**Rs.**	**21709.00**

5.) Comparison of Retrofitting Cost vis-à-vis the Cost of New Construction

Plinth Area Rate for New Construction =. 21709.00 /sqm

Total Area of Hospital considered = 5487.5 sqm.

Therefore the total cost of construction of new hospital of equivalent area = 5487.5 X 21709
= Rs. 119128137.5

Total cost for retrofitting old hospital comprising of structural, non-structural and miscellaneous works = Rs. 52651739.38

SPECIFIC ISSUES

1) The entire pitched roofing arrangement in Block-C needs to be replaced.

2) Since each truss in Block-B is in two separate parts and it would incur huge cost to rectify the same, hence it is advisable to replace the entire truss arrangement with inclined RCC roof.

3) Foundation – Not considered for load bearing structures as no mention in code.

4) Openings in masonry structure needs to be reduced to achieve the permissible level mentioned in the codes.

5) Doctors residences, Nurses Hostel, etc have not been considered for arriving at the retrofitting costs.

CHAPTER 4
SUMMARY & CONCLUSION

4.1 SUMMARY

Inadequacies of many un-reinforced masonry (URM) buildings have been realized in recent earthquakes in India and hence method of ensuring adequacy of such buildings is of urgent need. Although a considerable research is directed to study the reinforced concrete building, there is no structured methodology to assess the URM building in our country is available. It is important to develop systematic method of evaluation of existing URM buildings.

It is a general finding that masonry structures generally lacks bands due to which long structural crack / hair line crack develops which ultimately led structure loss of its load carrying capacity. Therefore their is need to break this continuity and provides bands at vlarious levels.

Their is not provision for lateral load resistance therefore shear walls must be additionally built which helps in providing seismic resistance to existing building to resist earthquake loads. Also it is seen that structure should be more of ductile nature to avoid sudden collapse of building. Their is also a great need to develop new IS codes as per latest seismic provision as that of R.C.C. structure to enhance the seismic behaviour of old masonry structure which accounts for more than 70% of building in India.

From on analysis we have found that bands (such as plinth, sill level, lintel, roof, gable), jacketing, epoxy grouting, fibre reinforced fabric, base isolation, steel bracing, friction damper, post tensioning and various mechanical anchors etc are very important retrofitting techniques.

4.2 CONCLUSION

IS 1905-1987 provides a semi empirical approach to the design of un reinforced masonry especially for stresses arising from vertical and moderate lateral loads, such as wind. The permissible stress values are not directly linked to the prism test values and do not address the strength and ductility of masonry members under large lateral loads due to earthquakes. Further use of reinforcement is necessary to improve its flexural resistance and ductility required for seismic loads. The masonry codes of other countries provide detailed provision for the design of reinforced masonry members.

IS : 1905 should be expanded to incorporate such provisions. The design approach in IS 1905-1987 is semi empirical, which combines allowable stress design with rules of thumb for unreinforced masonry only. Neither limit state methodology has been adopted in this code nor there are any provisions related to reinforced masonry for any design philosophies. Enhancements and modifications of IS : 1905-1987 is urgently required to address these issues.

Apart from these direct detailed analysis additionally, one can also go for non-destructive test and other indirect methods to check strength & durability & building such as rebound hammer, UPSV (Ultra Sonic Pulse velocity test), Abrasion test, penetration test, Half cell potential test, core cutting tests etc.

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