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Verification of adequacy of structural elements dimensions and Analytical Investigation of structural behaviour of Masonry Infilled and Ferrocement infilled G+2 storied existing structure and through Pushover analysis

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ABSTRACT

Pushover analysis of an existing structure located at Bangalore is carried out to evaluate its available strength. Analysis was carried out in different stages such as modelling for a bare frame structure and infilled with brick masonry and ferrocement with the consideration of openings such as doors and windows. The analysis outcome revealed that the few of the structural members was insufficient to carry external forces for various seismic zone intensities and demanded the retrofitting and thus revision of the structural dimensions were suggested and further analytical studies showed that base shear carrying capacity and displacement parameters significantly affects for different types of infilled materials and bare frames.
Keywords : Masonry Infill, Ferrocement, Pushover analysis

1. Introduction

A large number of existing buildings in India are severely deficient against earthquake forces and the number of such buildings are growing very rapidly. Retrofitting of any existing building is a complex task and requires skill, retrofitting of RC buildings is particularly challenging due to complex behaviour of the RC composite material. The behaviour of the buildings during earthquake depends not only on the size of the members and amount of reinforcement, but to a great extent on the placing and detailing of the reinforcement. There are three sources of deficiencies in a building, which have to be accounted for by the retrofitting engineer:

1) Inadequate design and detailing
2) Degradation of material with time and use
3) Damage due to earthquake or other catastrophe.

The three sources suggest a retrofit scheme to make up for the deficiencies and demonstrate that retrofitted structure will be able to safely resist the future earthquake forces expected during the lifetime of the structure. In particular, seismic rehabilitation of older concrete structures in high seismicity areas is a matter of growing concern, since such structures are vulnerable to damage must be identified and an acceptable level of safety must be determined.

Masonry infills (MI) within reinforced concrete frames are built with the assumption that these infills will not take part in resisting any kind of load either axial or lateral; hence its significance in the analysis of RC frame is generally neglected. Moreover, due to the non-availability of realistic and simple analytical models of MI becomes another hurdle for its consideration in analysis. In fact, an infill wall enhances considerably the strength and rigidity of the structure. It has been recognised that RC frames with MI have more strength and rigidity in comparison to the bared frames and the ignorance of MI has become the cause of failure of many of the multi-storeyed buildings. In masonry infilled structures the ordinarily occurring dead or live loads do not pose much of a problem, but the lateral loads due to wind and earthquake, tremors or blast loads are a matter of great concern and need special consideration in the design of buildings. These lateral forces can produce the critical stresses in a structure, set undesirable vibrations and in addition cause lateral sway of the structure.

Ferrocement is a relative a good material consisting of wire meshes and cement mortar. This material was developed by Nervi, an Italian Architect and Engineer, in 1940. It consists of closely spaced wire meshes which are impregnated with rich cement mortar mix. Ferro cement is a type of thin wall reinforced concrete, commonly constructed of hydraulic cement mortar, reinforced with closely spaced layers of continuous and relatively small size wire mesh. The mesh may be made of metallic or other suitable materials.

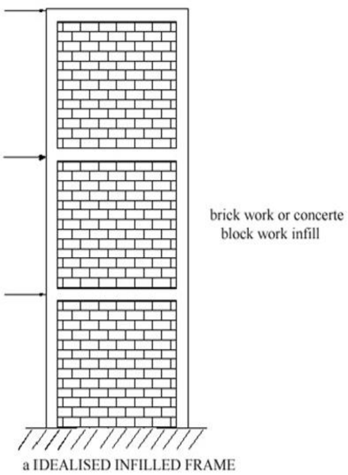
In order to evaluate the strength of the existing buildings or the newly proposed buildings, pushover tool serves as the best tool as the outcome of this analysis provides more of a realistic behaviour of the structure with considerably less estimated time compared to other dynamic analysis of the structures.

Here in this research an existing structure located in Bangalore is considered. Initially the existing structure details are collected and linear static analysis was carried to verify the dimensional stability of the various structural elements, in the later phase pushover analysis was carried to check the adequacy of the structural members subjected to various seismic zone intensities and lastly the structure was infilled with brick masonry and ferrocment along with the provisons of openings such as doors and windows and the parametric studies was carried out. A thorough literature survey on pushover analysis, infilled frames and ferrocement was carried out prior to beginning of this research work.

2. Infilled Frames

An infilled frames typically consists of a steel or RC frame with plain or reinforced brick masonry, block-work infilling (Figure 1a) in which restraint against lateral loads is provided by composite action of the infill and frame. The strength and energy dissipation capacity of an infilled frame is much higher than that of bare frame. It provides efficient and effective method of bracing to the building. It is effective because, the walls which are provided primarily for functional purposes are put into dual mode of acting, as partitions and enclosures bracing elements as well. It is efficient because composite in-plane action renders the structure stiff and strong. A frame with an infill wall is very effective against an earthquake, even though the input force increases because of the higher stiffness. However, these masonry walls cause stress concentration in particular members and /or deformation of the frame. In the recent past and it is proved that there is a strong interaction between the infill masonry wall and the surrounding frame leads to the composite frame behaviour, increased overall stiffness and in plane moment of inertia and also the considerable reduction of the probability of collapse.

Figure 1 (a) shows an idealized masonry infilled frame. Typical behaviour of a masonry infilled RC frame is show in Figure 1 (b). When deformed by lateral loads, the frame separates from the MI at various locations and in the remaining lengths of contact slip may occur. This leads to the concept of replacing the MI by an equivalent diagonal strut. Based on this concept analogous pin jointed RC frame shown in Figure 1(c) has been proposed by earlier investigators for the analysis of solid masonry infilled RC frames. The actual behaviour of a masonry infilled RC frame is more complex than that of simple pinned diagonal strut and is dependent on various parameters like aspect ratio of infill plane, relative stiffness of infilled frame, load ratio, material properties, support conditions etc. During the interaction between the two heterogeneous materials, due to differential deflection, there exists a physical separation at the interface. Depending on whether the physical separation at the interface is allowed to exist or not (it can be prevented by providing connectors between masonry infill and beam), bending moment and axial force in the beam and masonry infill wall stresses vay.



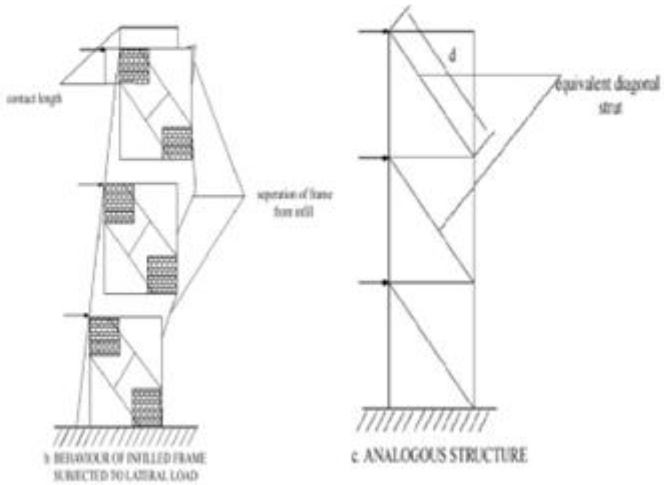


Figure 1 a) idealised infilled frame b) behaviour of infilled frame subjected to lateral load
c) Analogous structure

In a single bay frame, there are four surface of contact between the RC frame and beam/column. Due to various practical reasons, (like temperature variation differential deflection etc) any one or more of this surface may not be in contact with each other. Sometimes, to avoid bottommost storey from excessive stress, initial gaps are created between beams and masonry. Thus, presence (or absence) of surface contact at the four faces is an important parameter that affects the design parameter that affects the design parameters in the beams/columns and also masonry stresses. A relatively rigid and stronger material like ferrocement panel infill can be used to overcome the separation action as seen in the brick masonry since the ferrocement panel is a whole in a section separation may be very negligible. Efforts have been made in many research works for the last decade to exploit the inherent stiffness and strength of ferrocement used as the roof and floor material and now a days trend is raising to use ferrocement as infill's in multi-storeyed buildings . Now it is being recognized that ferrocement infill's are very effective in bracing the frames composed of columns and beams to resist in plane lateral loads. An acceptable design procedure has also been suggested for the design of infill frames without openings. A judicious location and spacing of infill frames in a building may permit the adoption of relatively smaller sections and less reinforcements leading to a reduction in the cost of reinforced concrete buildings. This is of greater significance to the countries like India where numerous medium size residential and office / business buildings are constructed usually of reinforced concrete frames with brick masonry infill which can be replaced by ferrocement panelled infill's to achieve faster, more durable, stiffer and economical constructions

3. Ferrocement

Ferro cement is a type of thin wall reinforced concrete, commonly constructed of hydraulic cement mortar, reinforced with closely spaced layers of continuous and relatively small size wire mesh. The mesh may be made of metallic or other suitable materials

3.1 Constituent Materials Ferrocement can be divided into two main components: the matrix and the reinforcement.

a.Matrix

The matrix is a hydraulic cement binder, which may contain fine aggregates and admixtures to control shrinkage and set time, and increase its corrosion resistance. The binder is itself a composite material consisting of a hydrated cement paste and an inert filler material.

b.Cement

Ordinary Portland cement can be utilized in the production of ferrocement.

c.Fine Aggregates

Normal weight fine aggregate (sand) is the most common aggregate used in ferrocement. It should be clean, hard, strong, and free of organic impurities and deleterious substances and relatively free of silt and clay. It should be inert with respect to other materials used and of suitable type with respect to strength, density, shrinkage and durability of the mortar made with it.

d.Reinforcement

The reinforcement of ferrocement is commonly in the form of layers of continuous mesh fabricated from an assembly of continuous single strands filaments. Specific mesh types include woven and welded mesh, expanded metal lath and perforated sheet products. There is a wide variety in mesh dimensions, as well as in the amounts, sizes and properties of the materials used . Figure 2 shows various types of reinforcing mesh.

e.Skeletal Steel

Skeletal steel used for making the framework of the structure upon which layers of mesh are laid. Both the longitudinal and transverse rods are evenly distributed and shaped to form. The rods are spaced as widely as possible up to 300mm apart where they are not treated as a structural reinforcement and are often considered to serve as spacer rods to the mesh reinforcements.

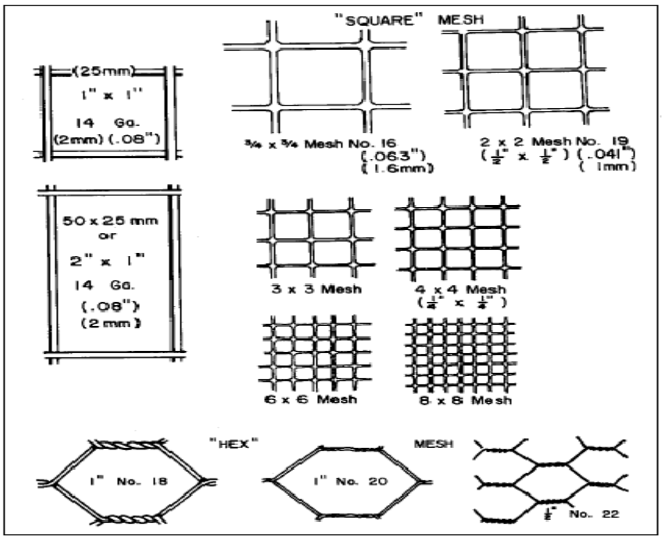


Figure 2 Reinforcing Mesh .

4. Push Over Analysis

The Non–Linear Static Procedure is defined in the Federal Emergency Management Agency document 356 (FEMA 356) as a non–linear static approximation of the response a structure will undergo when subjected to dynamic earthquake loading. The static approximation consists of applying a vertical distribution of lateral loads to a model which captures the material non–linearity of an existing or previously designed structure, and monotonically increasing those loads until the peak response of the structure is obtained on a base shear vs. roof displacement plot as shown in Figure 3.

Pushover analysis is carried out to develop a capacity curve for the building. Based on the capacity curve, a target displacement which is an estimate of the displacement that the design earthquake will produce on the building is determined. The extent of damage experienced by the structure at this target displacement is considered representative of the damage experienced by the building when subjected to design level ground shaking. Many methods were presented to apply the non- linear static pushover (NSP) to structures. These methods can be listed as;

1) Capacity spectrum method (CSM)
2) Displacement coefficient method (DCM)
3) Modal Pushover analysis (MPA)

In general, analytical models for the pushover analysis of frame structures may be divided into two main types namely distributed plasticity (plastic zone) and Concentrated plasticity (plastic hinge)

Although the plastic hinge approach is simpler than the plastic zone, this method is limited to its incapacity to capture the more complex member behaviour that involve severe yielding under the combined actions of compression and bi-axial and buckling effects.

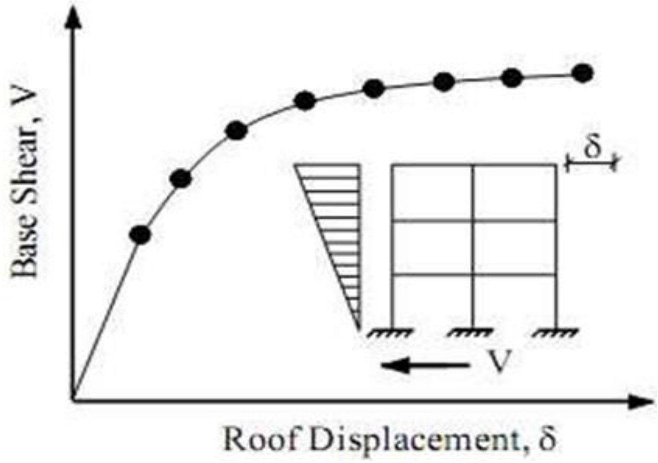


Figure 3 Global Capacity (Pushover) Curve of a Structure

Pushover analysis can be performed as force-controlled or displacement- controlled. In force-controlled pushover procedure, full load combination is applied as specified, i.e.; force-controlled procedure should be used when the load is known (such as gravity loading). Also, in force-controlled pushover procedure some numerical problems that affect the accuracy of results occur since target displacement may be associated with a very small positive or even a negative lateral stiffness because of the development of mechanisms and P-delta effects. In displacement-controlled procedure, specified drifts are sought where the magnitude of applied load is not known in advance. The magnitude of load combination is increased or decreased as necessary until the control displacement reaches a specified value. Generally, roof displacement at the center of mass of structure is chosen as the control displacement. The internal forces and deformations computed at the target displacement are used as estimates of inelastic strength and deformation demands that have to be compared with available capacities for a performance check.

4.1 Necessity of Non Linear Static Pushover Analysis

The existing building can become seismically deficient since seismic design code requirements are constantly upgraded and advancement in engineering knowledge. Further, Indian buildings built over past two decades are seismically deficient because of lack of awareness regarding seismic behaviour of structures. The widespread damage especially to RC buildings during earthquakes exposed the construction practices being adopted around the world, and generated a great demand for seismic evaluation and retrofitting of exixting building stocks.

4.2 Purpose of Non-Linear Static Pushover Analysis

The purpose of pushover analysis is to evaluate the expected performance of structural systems by estimating performance of a structural system by estimating its strength and deformation demands in design earthquake by means of static inelastic analysis and comparing these demands to available capacities at the performance levels of interest. The Performance is based on assessment of important performance parameters, including global drift, inter story drift, inelastic element deformations (either absolute or normalized with respect to yield value) deformations between elements and element connection forces. The inelastic static push over analysis can be viewed as a method for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces that no longer can be resisted within the elastic range of structural behaviour. The push over is expected to provide below information on many response characteristics that cannot be obtained from an elastic static or dynamic analysis.

- Identification of location of weak points in the structure(or potential failure modes)
- Identification of the critical regions in which the deformation demands are expected to be high and that have to become the focus through detailing.
- Determination of force demands on brittle members, such as axial force demands on columns, moment demands on beam-column connections.
- The realistic force demands on potentially brittle elements, such as axial force demands on columns, force demands on brace connection, moment demands on beam to column connections, shear force demands in deep reinforced concrete spandrel beams, shear force demands in unreinforced masonry wall peirs,etc.
- Estimates of the deformations demands for elements that have to form in elastically in order to dissipate the energy imparted to the structures.
- Consequences of the strength deterioration of individual elements on behaviour of structural system.
- Identification of the strength discontinuities in plan elevation that will lead to changes in the dynamic characteristics in elastic range.
- Estimates of the interstorey drifts that account for strength or stiffness discontinuities and that may be used to control the damages and to evaluate P-Delta effects.
- Verification of the completeness and adequacy of load path, considering all the elements of the structural system, all the connections, the stiff non-structural elements of significant strength, and the foundation system.

4.3 Evaluation of the Performance Level

Performance evaluation is the main objective of a performance based design. A component or action is considered satisfactory if it meets a prescribed performance .The main output of a pushover analysis is in terms of response demand versus capacity. If the demand curve intersects the capacity envelope near the elastic range, then the structure has a good resistance. If the demand curve intersects the capacity curve with little reserve of strength and deformation capacity, then it can be concluded that the structure will behave poorly during the imposed seismic excitation and need to be retrofitted to avoid future major damage or collapse.

4.4 Performance Levels and Ranges

The desired condition of the structure after a range of ground shakings, or building performance level, is decided by structural engineer. The building performance level is a function of the post event conditions of the structural and non-structural components of the structure. The performance levels as per FEMA 356 are as follows:

a) Immediate Occupancy
b) Life Safety
c) Collapse Prevention

It can be observed from the Figure 4 shown below table that for the three building performance levels of Operational, Immediate Occupancy and Life Safety, due regard has to be given to both structural and non-structural performance levels. For the building performance level of Collapse prevention, the performance of the non-structural component can be neglected.

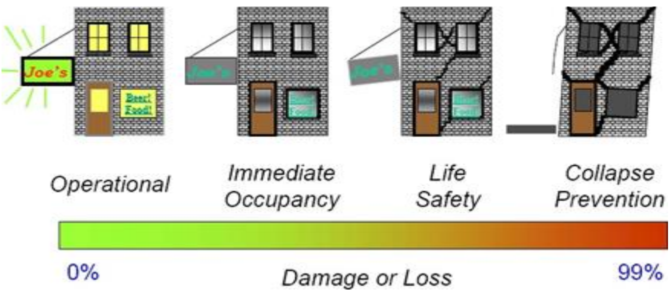


Figure 4 Performance Levels

4.5 Pushover Analysis using ETABS

ETABS is a special purpose computer program developed specifically for building systems. Creating and modifying a model, executing the analysis, design, and optimizing the design are all done through this single interface. The analytical capabilities of ETABS are just as powerful, representing the latest research in numerical techniques and solution algorithms. It is a versatile and user-friendly program that offers a wide scope of features like static and dynamic analysis, nonlinear dynamic analysis and nonlinear static pushover analysis, etc. These features and many more, make ETABS the state-of-the-art in structural analysis programs. ETABS Nonlinear - Includes all of the capabilities of ETABS including nonlinear static and dynamic analysis. Three-dimensional static nonlinear (pushover) analysis for uniform load patterns, load patterns based on mode shapes, arbitrarily defined load pattern is possible. Nonlinear hinge property definition data is set up such that both user-defined hinge properties and the hinge properties designated in the ATC- 40 and FEMA-273 documents can be easily assigned. Capacity spectrum analysis is automatically performed and graphical a well as printed output is provided.

4.6 Developing Pushover Curve

After assigning all properties of all the models, the displacement controlled pushover analysis of models are carried out. The models are pushed in monotonically increasing order until target displacement is reached or structure loses equilibrium whichever occurs first. For this purpose target displacement at roof level and number of steps in which this displacement must be defined. Pushover curve is a base shear force versus roof displacement curves. The peak of this curve represents maximum lateral load carrying capacity of the structure. The initial stiffness of the structure is obtained from the tangent at pushover curve at zero load level. The collapse is assumed when structure losses its 75% strength and corresponding roof displacement is called ''maximum roof displacement''. It is a plot drawn between base shear and roof displacement. Performance point and location of hinges in various stages can be obtained from pushover curve. The range AB in elastic range, B to IO is the range of immediate occupancy IO to LS is the range of life safety and LS to CP is the range of collapse prevention. When a hinge reaches point C on its force displacement curve that hinge must begin to drop load. The way load is dropped from a hinge that has reached point C is that the pushover force (base shear) is reduced until the force in that hinge is consistent with the force at point D. As the force is dropped, all elements unload, and the displacement is reduced. Once the yielded hinge reaches the point D force level, the pushover force is again increased and the displacement begins to increase again. If all the hinges are within the CP limit then the structure is said to be safe. However, depending upon the importance of structure the hinges after IO range may also need to be retrofitted.

5. Scope of the Study

The main focus of the present work is to carry out comparative study on a RC building. For this purpose frames are modelled using ETABS, further the performance under non linear incremental loading is studied based on the pushover curves and capacity-demand curves generated up to maximum displacement. The study includes the consideration of effect of base shear and displacement for two storey RC bare frame, bare frame infilled with brick and ferrocement.

The following three different cases of models are considered namely

1 Bare frame analysis for gravity loads and all the seismic zones of India

2 Brick masonry infilled frame with openings gravity loads and all the seismic zones of India

3 Ferrocement infilled frame with openings for seismic zone V.

6. Structural Model Description

In this study an existing RC brick infilled building located at Ramachandrapura, near BEL circle, Bangalore have been considered. A brief summary of the building is presented in the Table 1. The plan at ground, first and second storey along with front elevation of the building is as shown in the figure 5. Figure 6 shows the dimension measurements of column and beam.

Table 1 Details of the structural model

1	Type of structure	Ordinary moment resisting RC frame
2	Grade of Concrete	M 20
3	Young's modulus	22360 MPa
4	Grade of Reinforcing Steel	Fe 415
5	Number of stories	G+2
6	Building height	9m above the ground storey
7	*Column size*	Rectangular columns at the plinth, ground floor and first floor level: 150mmx450mm
8	Beam size	Rectangular beam at the ground floor and first floor level: 150mmx450mm
9	Slab Thickness	*The concrete slab is 150 mm thick at each floor level.*
10	*Structural system*	The building is an RC framed structure. The floor plan is same for all floors.
11	*Foundation*	The structure is resting on isolated footing.
12	*Dead load and*	As per IS 875 Part 1
13	*Live load*	Roof level - 1.5kN/m^2 Floor level- 2 N/m^2 On Balcony- 3 N/m^2
14	*Seismic forces*	A per IS 1893

6.1 Structural modelling

The analytical model was created in such a way that the different structural components represent as accurately as possible the characteristics like mass, strength, stiffness and deformability of the structure. The various primary structural components that were modelled are as follows:

(a) Beams and columns: Beams and columns were modelled as 3D frame elements. The members were represented through the assignment of properties like cross sectional area, reinforcement details and the type of material used. Figure 6 gives the details of beam and column dimension measurement at the site.

(b) Beam-column joints: The beam-column joints were assumed to be rigid and were modelled by giving end-offsets to the frame elements. This was intended to get the bending moments at the face of the beams and columns.

(c) Foundation modelling: The foundation was modelled based on the degree of fixity which is provided. The effect of soil structure interaction was ignored in the analysis. In the model, fixed support was assumed at the column ends at the end of the footing.

(d) Slab Modelling (Modelling of joints): Slab is modelled as a rigid diaphragm. In rigid diaphragm case all the joints in the slab moves together as a single unit. Being a rectangular slab meshing was done by dividing the area into smaller rectangular segments. Meshing improves the results but increases the computational time by a large extent.

(e) Wall element: Wall element was modelling with the feature available in Etabs by defining the suitable material propert for both brick masonry and ferrocement infill. The details required to model ferrocement infill was obtained from an experimental analysis and the structural element was created and assigned. At appropriate places openings was provided based on the plan details. Meshing of the wall is done. Table 2 provides the details of openings.

Table 2 Opening sizes of the structural model

Opening	**Sizes**
Window	1.83 X 2m
Window-1	1.83 X1.403m
Window-2	1.22 X1.403m
Window-3	1.83 X1.915m
Window-4	0.915X 1.403m
Ventilator	0. 3X0.9m
Main door	1.098X 2.135m
Door	0.915X 2.135m
Door- 1	0.8X 2.135m

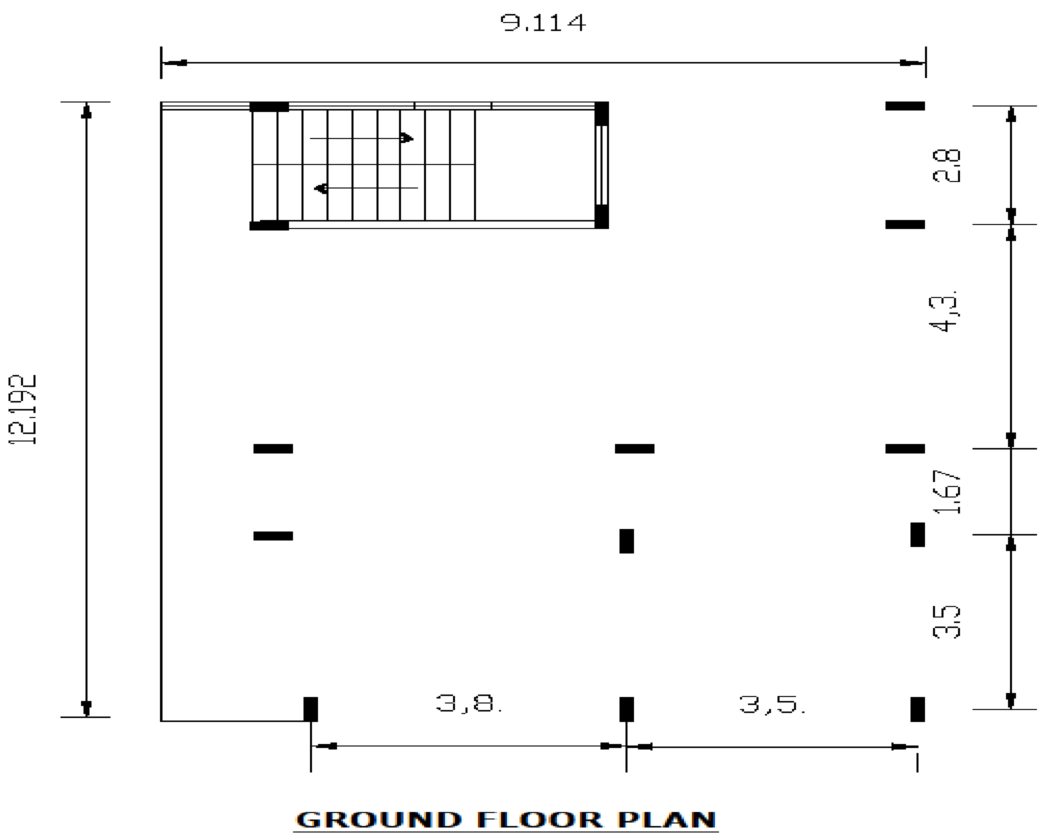


Figure 5a Ground floor Plan

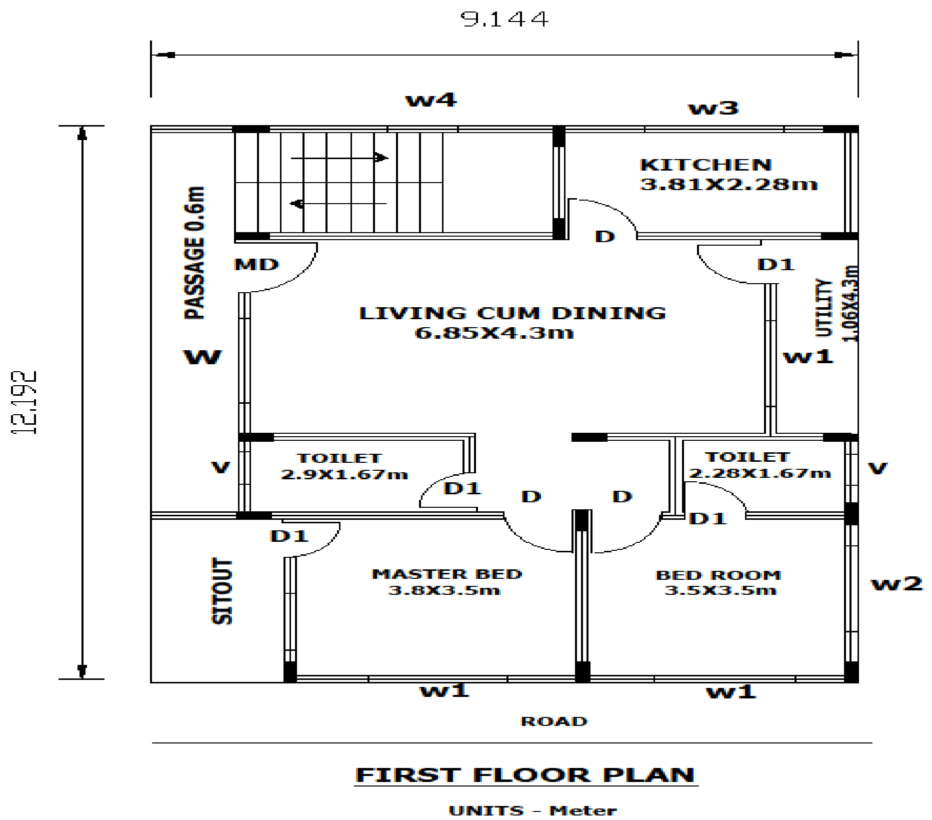


Figure 5b First floor Plan

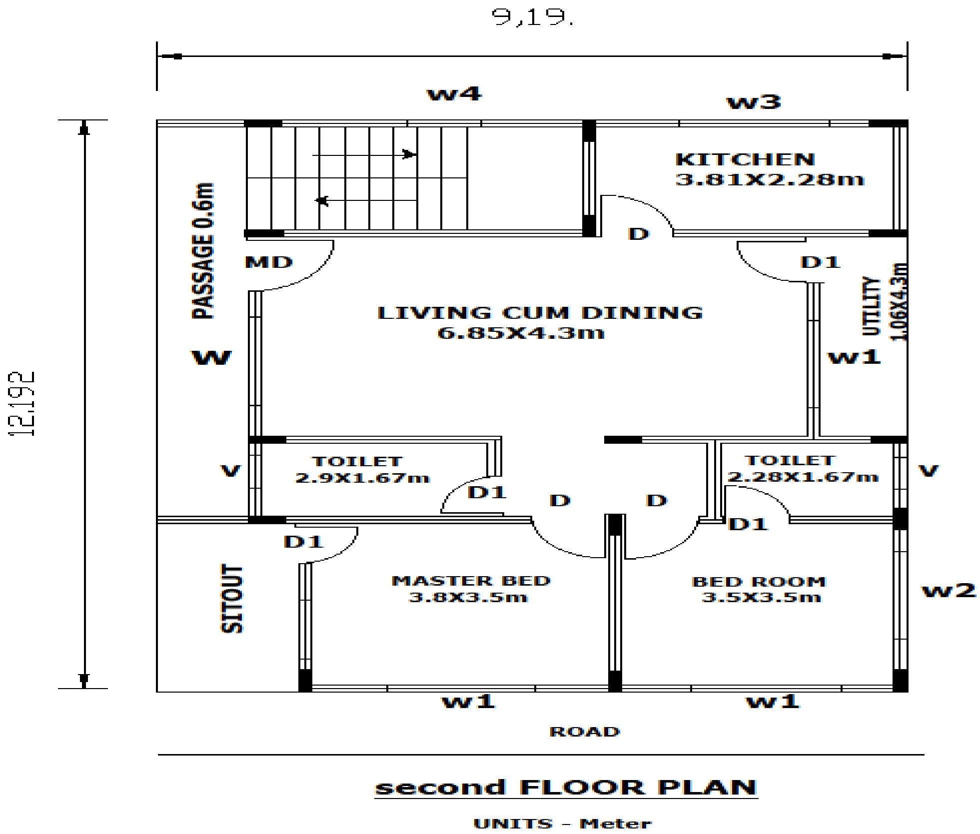


Figure 5c Second floor Plan

Figure 5d Front View of the building Plan

Figure 6a Width of existing building column 150mm

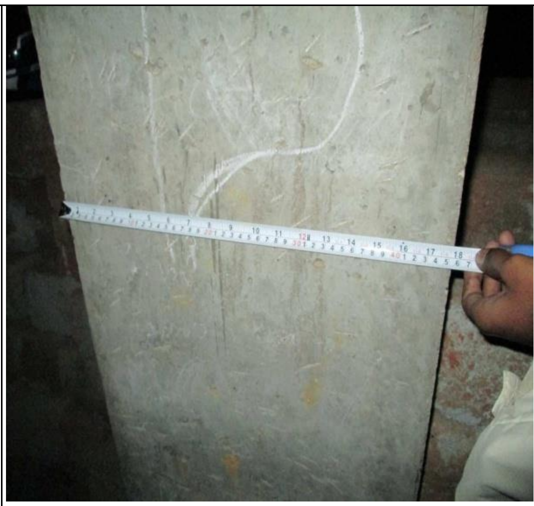


Figure 6b Depth of existing building column 450mm

Figure 6c Width of existing beam 150mm

In this study totally 11 number of models were created. Out of which 5 cases each of bare frames and Brick infilled frames for different loading conditions which is presented in the Table 3. Table 4 and Table 5 provides the various details considered for modelling aspects and structural details for bare frame and brick infilled frame respectively.

Table 3 Model description details

1	For Bare frame analysis	Case A- Gravity Loads
		Case B- Sesimic zone 2
		Case C- Sesimic zone 3
		Case D- Sesimic zone 4
		Case E- Sesimic zone 5
2	Brick Infilled frames	Case A- Gravity Loads

		Case B- Sesimic zone 2
		Case C- Sesimic zone 3
		Case D- Sesimic zone 4
		Case E- Sesimic zone 5

Table 4 Case I: Bare frame

	Case A	**Case B (Zone-II)**	**Case C (Zone-III)**	**Case D (Zone-IV)**	**Case E (Zone-V)**
Loads considered	DL&LL	DL,LL&EQ	DL,LL&EQ	DL,LL&EQ	DL,LL&EQ
Zone factor(Z)	-	0.10	0.16	0.24	0.36
Importance factor(I)	-	1	1	1	1
Response reduction factor (R)	-	3	3	3	3
Soil type	-	II	II	II	II
Time period(T)	-	0.389sec	0.389sec	0.389sec	0.389sec
Beam cross section(mm)	150X450	150X450	150X450	150X450	150X450
Column cross section(mm)	150X450	150X450	150X450	150X450	150X450
Slab thickness(mm)	150	150	150	150	150

Table 5 (Case II: Brick infill with openings)

	Case A	**Case B (Zone-II)**	**Case C (Zone-III)**	**Case D (Zone-IV)**	**Case E (Zone-V)**
Loads considered	DL&LL	DL,LL&EQ	DL,LL&EQ	DL,LL&EQ	DL,LL&EQ
Zone factor(Z)	-	0.10	0.16	0.24	0.36
Importance factor(I)	-	1	1	1	1
Response reduction factor (R)	-	3	3	3	3
Soil type	-	II	II	II	II

Time period(T) T_x	-	0.23sec	0.23sec	0.23sec	0.23sec
Time period(T) T_y	-	0.26sec	0.26sec	0.26sec	0.26sec
Beam (mm)	150X450	150X450	150X450	150X450	150X450
Column (mm)	150X450	150X450	150X450	150X450	150X450
Slab thickness(mm)	150	150	150	150	150

Linear static analysis is carried out for all the above said cases in order to find out whether the existing dimensions are sufficient to carry the dead load, live load and seismic loads for various zones. Table 6 provides the analytical results for case A of bare frames subjected to gravity loads. It is noticed that fewer beams have failed and no columns have failed.

Table 6 Parameters considered and Analysis results for Case A Bare frames

Case A		**Analysis Results**	
Loads considered	DL&LL	BEAMS	COLUMNS
Beam (mm)	150X450	Beam no 18 failed at Storey3, storey2	None of the columns failed.
Column (mm)	150X450	Storey1 and at plinth level.	
Slab thickness(mm)	150		

Case B bare frame analysis results are shown in table 7 and can be observed that significant number of beams have failed and all the columns have passed the seismic force of zone II intensity. As the intensity of the seismic forces are increased to zone III & IV more number of beams tends to fail and columns continue to perform well with no failures which could be noticed in the table 8 and table 9. However, in the seismic zone V significant number of columns also failed in addition to the beams as observed in the analysis results from table 10 and hence it can be concluded at this level of linear elastic analysis that the existing dimensions of beams and columns are inadequate to sustain the seismic forces and it demands for the rehabilitation or retrofitting.

Table 7 Parameters considered and Analysis results for Case B Bare frames

Case B(Zone II)- Parameters considered		**Analysis Results**	
Loads	DL,LL&EQ	Beams	Columns
Zone factor(Z)	0.10	B17 & B18 failed at Storey 3 B17 & B18 failed at Storey 2 B21 failed at storey 2 B17 & B18 failed at Storey 1 B21 failed at Storey 2 B24 failed at Storey 1	None of the columns failed
Importance factor(I)	1		
Response reduction factor (R)	3		
Soil type	II		
Time period(T)	0.389sec		
Beam (mm)	150X450		
Column (mm)	150X450		
Slab thickness(mm)	150		

Table 8 Parameters considered and Analysis results for Case C Bare frames

Case C (Zone III)- Parameters considered		**Analysis Results**	
Loads	DL,LL&EQ	Beams	Columns
Zone factor(Z)	0.16	B17 & B18 failed at Storey 3 B17, B21, B24 & B18 failed at Storey 2 B20, B17, B18, B20, B21 & B24 failed at storey 1	None of the columns failed
Importance factor(I)	1		
Response reduction factor (R)	3		
Soil type	II		
Time period(T)	0.389sec		
Beam (mm)	150X450		
Column (mm)	150X450		
Slab thickness(mm)	150		

Table 9 Parameters considered and Analysis results for Case D Bare frames

Case D (Zone IV)- Parameters considered		**Analysis Results**	
Loads	DL,LL&EQ	Beams	Columns
Zone factor(Z)	0.24	B17 ,B18, B21 &	None of the columns

Importance factor(I)	1	B24 failed at Storey 3 B17, B18, B20, & B21, failed at Storey 2 B5, B6, B17, B18, B20 & B24 failed at storey 1	failed
Response reduction factor (R)	3		
Soil type	II		
Time period(T)	0.389sec		
Beam (mm)	150X450		
Column (mm)	150X450		
Slab thickness(mm)	150		

Table 10 Parameters considered and Analysis results for Case E Bare frames

Case E (Zone V)- Parameters considered		Analysis Results	
Loads	DL,LL&EQ	Beams	Columns
Zone factor(Z)	0.36	B17 ,B18, B20, B21 & B24 failed at Storey 3 B3, B5, B11,B18,B24 & B17, failed at Storey 2 B3,B5, B11, B17, B18, B20, B24 & B21 failed at storey 1 B5 Beam failed at plinth level	C2 ,C3 , C7 ,C6 ,C5 & C4 columns failed at Storey 2. C4 , C5, C6 ,C7 & C15 columns failed at Storey 1.
Importance factor(I)	1		
Response reduction factor (R)	3		
Soil type	II		
Time period(T)	0.389sec		
Beam (mm)	150X450		
Column (mm)	150X450		
Slab thickness(mm)	150		

Table 11 provides the details of parameters considered and analytical results of the Case A- Brick infilled frames. It is noticed in the results that no columns have failed and fewer beams have failed. When the same structure is subjecting the various seismic intensities of India, beams have failed for the seismic zone intensities of zone II, III & IV and columns have passed as shown in the table 12, table 13 and 14. Where as in the seismic zone V fewer columns as well as beams have failed even with the presence of brick infills as shown in table 15 and hence the sectional dimensions which are existing is highly inadequate to withstand gravity and lateral forces which is alarming the designers or practicing Engineers to be cautious about the structural behaviour before finalizing the structural elements dimensions.

Table 11 Parameters considered and Analysis results for Case A Brick Infilled frames

Case A		Analysis Results	
Loads considered	DL&LL	BEAMS	COLUMNS
Beam (mm)	150X450	Beam no 18 failed at Storey3, storey2	None of the columns failed.
Column (mm)	150X450	Storey1 and at plinth level.	
Slab thickness(mm)	150		

Table 12 Parameters considered and Analysis results for Case B Brick Infilled frames

Case B(Zone II)- Parameters considered		Analysis Results	
Loads	DL,LL&EQ	Beams	Columns
Zone factor(Z)	0.10	None of the beams failed	None of the columns failed
Importance factor(I)	1		
Response reduction factor (R)	3		
Soil type	II		
Time period(T)	0.389sec		
Beam (mm)	150X450		
Column (mm)	150X450		
Slab thickness(mm)	150		

Table 13 Parameters considered and Analysis results for Case C Brick Infilled frames

Case C(Zone III)- Parameters considered		Analysis Results	
Loads	DL,LL&EQ	Beams	Columns
Zone factor(Z)	0.16	None of the beams failed	None of the columns failed
Importance factor(I)	1		
Response reduction factor (R)	3		
Soil type	II		

Time period(T)	0.389sec		
Beam (mm)	150X450		
Column (mm)	150X450		
Slab thickness(mm)	150		

Table 14 Parameters considered and Analysis results for Case D Brick Infilled frames

Case D(Zone IV)- Parameters considered		**Analysis Results**	
Loads	DL,LL&EQ	Beams	Columns
Zone factor(Z)	0.24	B17 at Storey 1 and B5 at Plinth level have been failed.	None of the columns failed
Importance factor(I)	1		
Response reduction factor (R)	3		
Soil type	II		
Time period(T)	0.389sec		
Beam (mm)	150X450		
Column (mm)	150X450		
Slab thickness(mm)	150		

Table 15 Parameters considered and Analysis results for Case E Brick Infilled frames

Case E (Zone V)- Parameters considered		**Analysis Results**	
Loads	DL,LL&EQ	Beams	Columns
Zone factor(Z)	0.36	B17 & B21 beams have been failed at Storey 1. B3 ,B5, B11 & B24 beams have been failed at Plinth level.	C3 , C15 ,C4 , C5 , C6 & C7columns have failed at Storey 1. C13 ,C14 , C3& C4 columns have failed at Plinth level.
Importance factor(I)	1		
Response reduction factor (R)	3		
Soil type	II		
Time period(T)	0.389sec		
Beam (mm)	150X450		
Column (mm)	150X450		
Slab thickness(mm)	150		

Though the earthquake philosophy of strong column and weak beam is observed, it is clear from the results that the cross section of the Beams and columns are insufficient to carry the Dead and live loads (case A). From the overall observation of all the cases both bare frames and brick infilled frame it is understood that, the cross section of beam is highly insufficient to carry loads. Therefore it is necessary to revise the cross section of beam in order to make the structure stable. The revised cross section of beam is 230mmX450mm. After revising the cross section of beams and columns, linear static analysis was carried out. Results showed that none of the beams and columns have failed for all the above cases after dimension modifications which could be noticed in the Table 16.

Table 16 Analysis results after revising the structural dimensions for bare frame and brick infilled frame

Case I- Bare Frame	**Case II- Brick Infilled frame with openings**	**Analysis results for Beams**	**Analysis results for Column**
Case A (Beam Size 200x450mm)	Case A (Beam Size 200x450mm)	None of the beams have failed	None of the Columns failed
Case B (Beam Size 200x450mm)	Case B (Beam Size 200x450mm)		
Case C (Beam Size 200x450mm)	Case C (Beam Size 200x450mm)		
Case D (Beam Size 200x450mm)	Case D (Beam Size 200x450mm)		
Case E (Beam & Column size 200x450mm)	Case E (Beam & Column size 200x450mm)		

After verifying the dimensional stability of the structural model, for the revised structural dimensions of beams and columns, pushover analysis was carried out for bare frame, brick infilled frame and ferrocement infilled frame for the seismic zone 5 conditions. Pushover curves, capacity spectrum curves and hinge formation have been studied. Results are discussed in the subsequent sections. Table 17, table 18 and table 19 provides the details of pushover analysis results with respect to base shear, displacement and the number of hinges formed in the various performance levels and ranges.

Table 20 Maximum displacement and Base shear .

Sl no	Type	Maximum displacement(m)	Maximum Baseforce(KN)
1	Bare Frame	0.088	1070.94
2	Ferrocement Infill	0.0689	1164.80
3	Brick Infill	0.0629	1224.941

Table 20 provides the details of maximium base force and displacement for bare frame, brick and ferrocement infilled frame. It is found that brick infilled structure possess maximum baseforce compared to ferrocement infilled and bare frame structure. The least displacement is found be with brick infilled structure. The graphical representation of the same is shown in Figure 11 and figure 12.

Table 17 Results Of Pushover Curve For RC Bare Frame

Step	Displacement	Base Force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	TOTAL
0	0.0000	0.0	337	1	0	0	0	0	0	0	338
1	0.0083	382.6	304	34	0	0	0	0	0	0	338
2	0.0174	675.1	280	58	0	0	0	0	0	0	338
3	0.0266	836.1	261	70	7	0	0	0	0	0	338
4	0.0362	925.1	249	59	30	0	0	0	0	0	338
5	0.0457	981.4	240	56	38	4	0	0	0	0	338
6	0.0560	1014.5	237	45	20	36	0	0	0	0	338
7	0.0718	1042.4	235	45	13	45	0	0	0	0	338
8	0.0812	1058.9	232	48	10	47	0	1	0	0	338
9	0.0880	1070.9	232	48	9	45	0	1	3	0	338
10	0.0686	202.8	338	0	0	0	0	0	0	0	338

Table 18 Results of Pushover Curve for Brick Infilled Frame

Step	Displacement	Base force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	TOTAL
0	0	0	686	2	0	0	0	0	0	0	688
1	0.0054	512.3889	665	23	0	0	0	0	0	0	688
2	0.0103	848.6581	647	41	0	0	0	0	0	0	688
3	0.0138	967.5823	638	50	0	0	0	0	0	0	688
4	0.0161	1009.432	628	32	28	0	0	0	0	0	688

5	0.0258	1077.779	621	17	49	1	0	0	0	0	688
6	0.0354	1127.437	619	12	33	24	0	0	0	0	688
7	0.0481	1177.453	615	10	17	44	0	2	0	0	688
8	0.0629	1224.941	615	10	17	42	0	2	2	0	688
9	0.0629	1143.679	615	10	16	43	0	0	4	0	688
10	0.0629	1088.422	615	10	16	42	0	1	4	0	688
11	0.0633	1094.703	615	10	16	42	0	1	4	0	688
12	0.0488	-243.144	688	0	0	0	0	0	0	0	688

Table 19 Results Of Pushover Curve For Ferrocement InfilledFrame

Step	Displacement	Base force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	TOTAL
0	0	0	688	0	0	0	0	0	0	0	688
1	0.0033	250.9823	673	15	0	0	0	0	0	0	688
2	0.0104	730.4288	649	39	0	0	0	0	0	0	688
3	0.0142	872.8096	636	51	1	0	0	0	0	0	688
4	0.0193	975.9398	627	55	6	0	0	0	0	0	688
5	0.0234	1013.003	619	36	33	0	0	0	0	0	688
6	0.032	1054.992	615	19	52	2	0	0	0	0	688
7	0.0418	1088.215	612	19	33	24	0	0	0	0	688
8	0.0518	1117.921	610	19	23	36	0	0	0	0	688
9	0.0628	1149.434	608	19	13	46	0	2	0	0	688
10	0.0689	1164.804	608	19	12	45	0	1	2	1	688
11	0.0603	471.6875	688	0	0	0	0	0	0	0	688

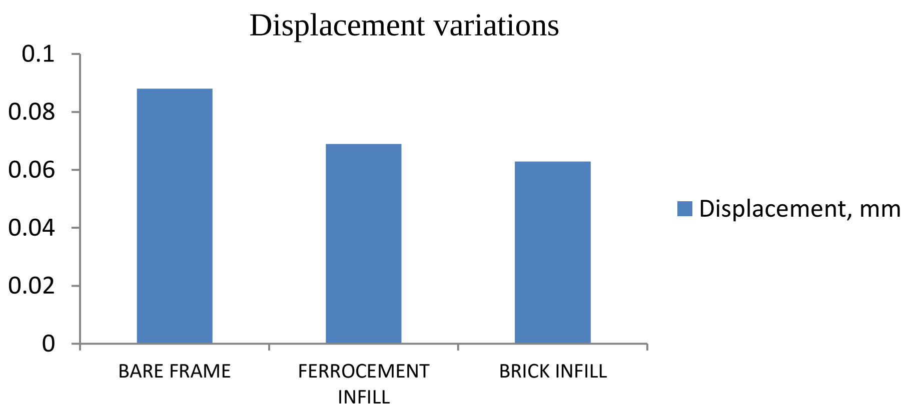


Figure 10 Displacement variations for structural models

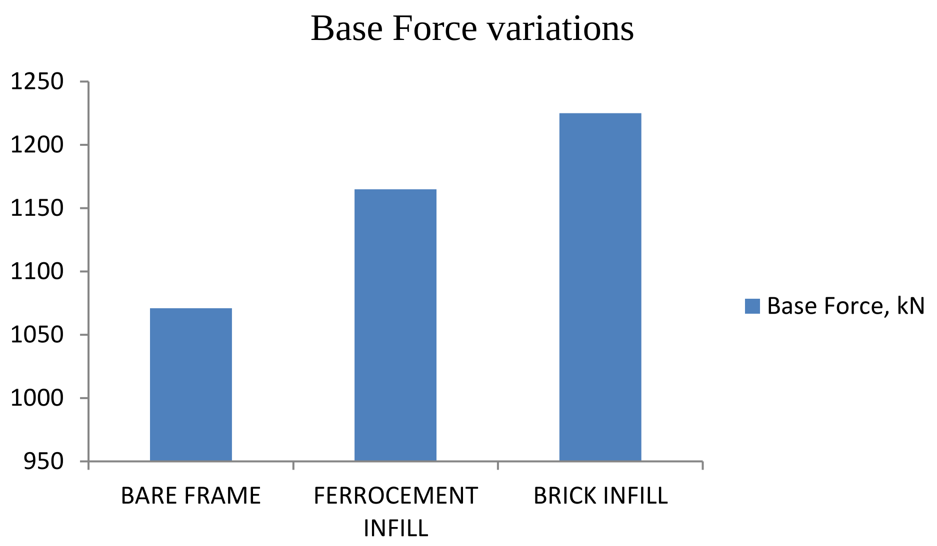


Figure 11 Base force variations for structural models

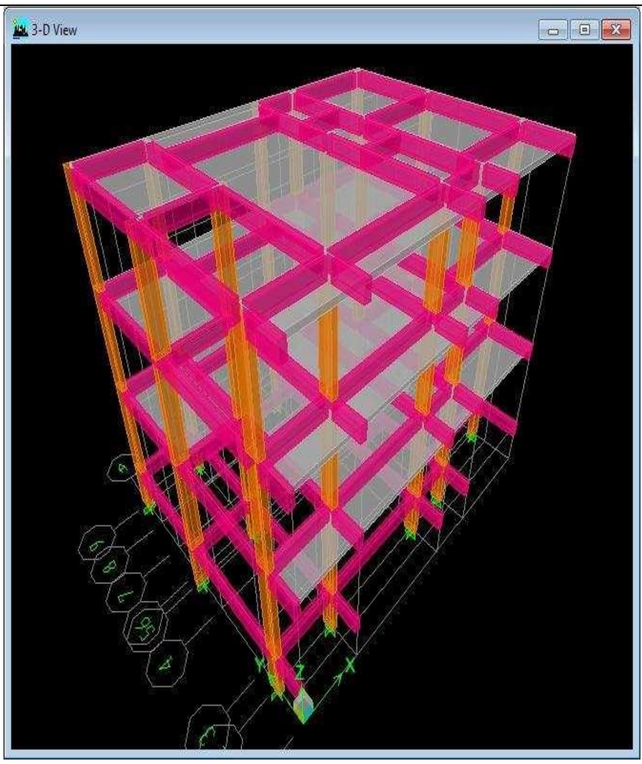


Figure 7a model of RC bare frame Etabs. 3D view

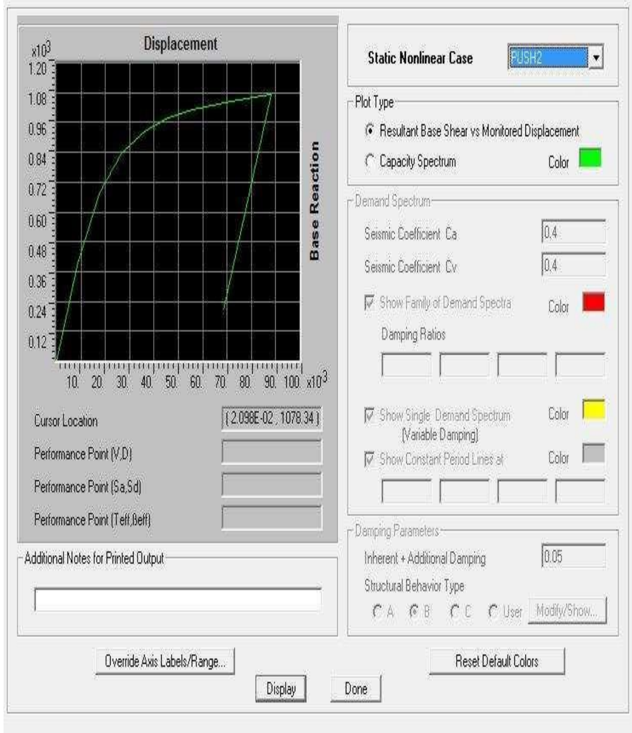


Figure 7b Pushover curve obtained for RC bare fram

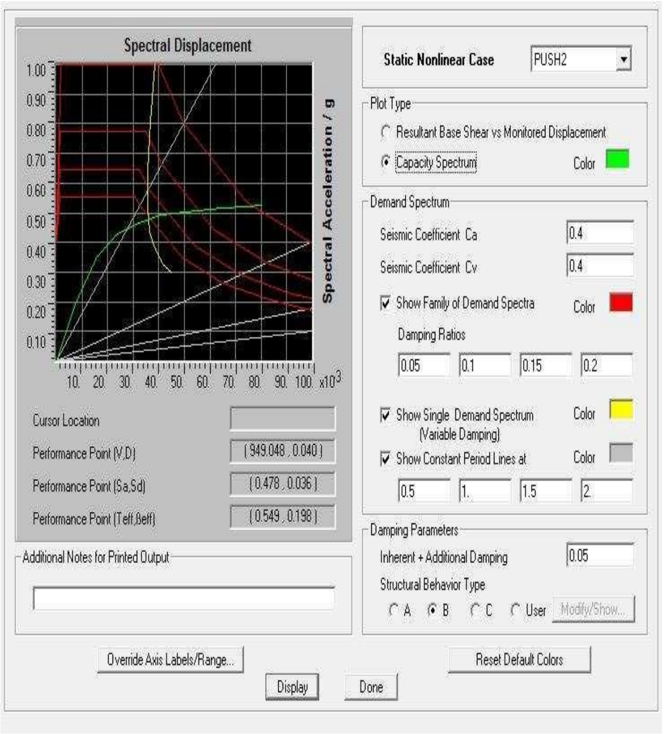


Figure 7c Capacity Demand Spectrum Obtained for RC Bare Frame

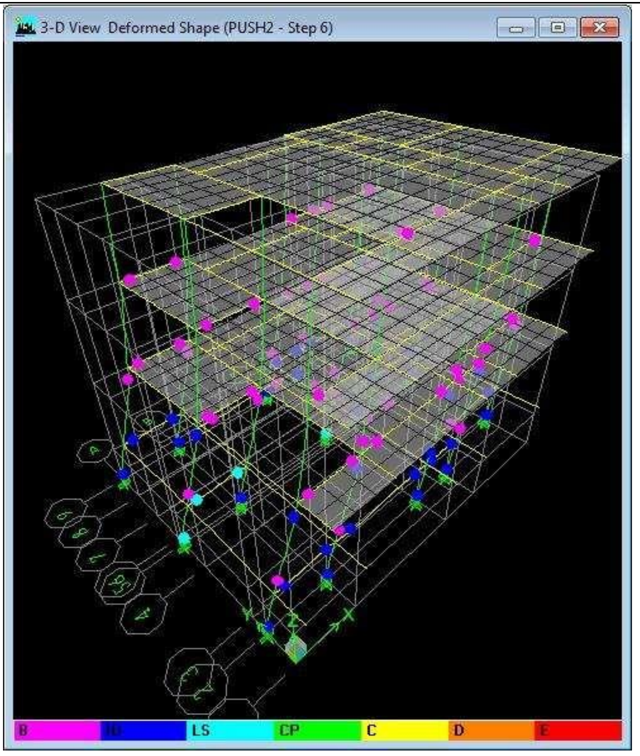


Figure 7d Hinge Levels Obtained for RC Bare Frame

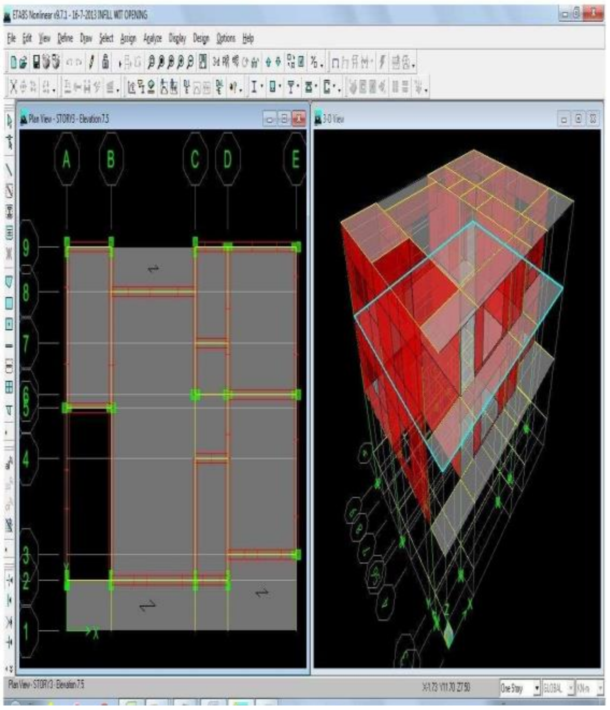


Figure 8a RC Frame With Brick As Infill

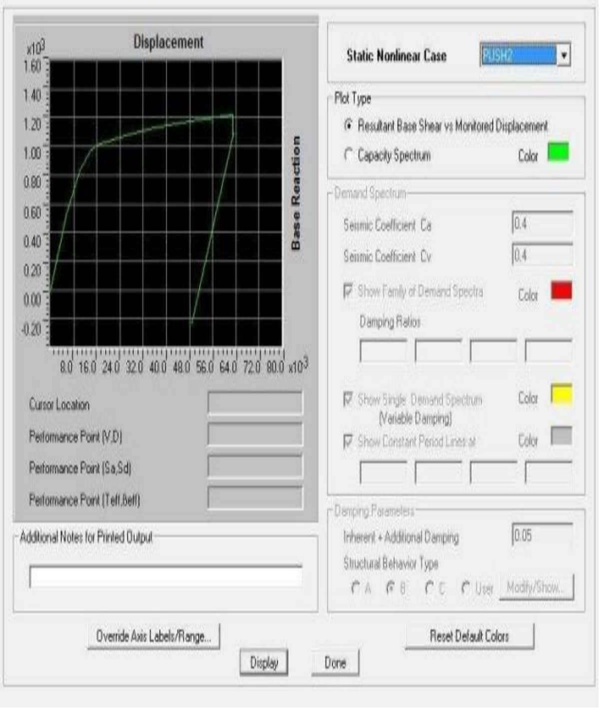


Figure 8b Pushover Curve Obtained for RC Frame With Brick Infill

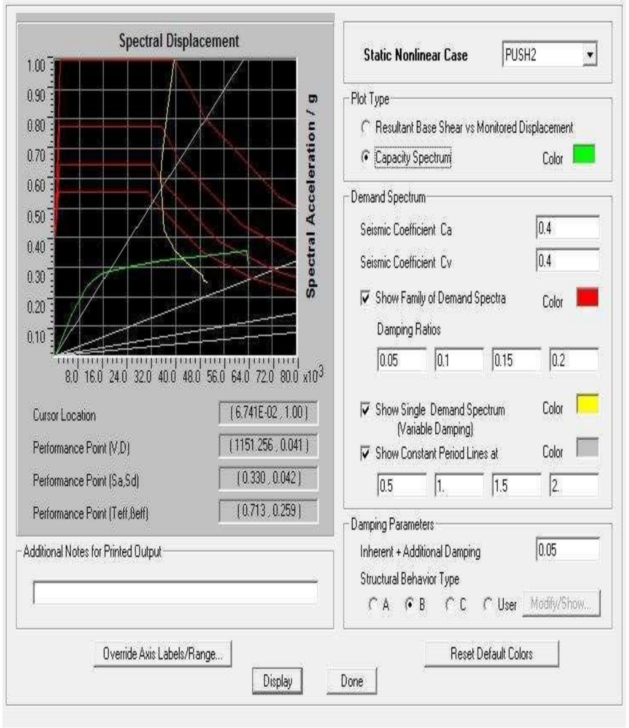


Figure 8c Capacity Demand Spectrum Obtained For Rc Frame WithBrick Infil

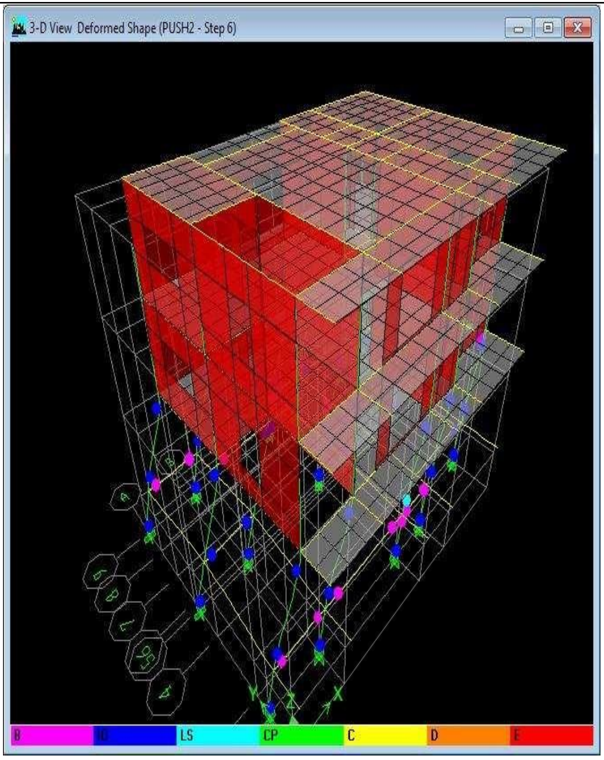


Figure 8c Hinge Levels Obtained For Rc Frame With Brick Infill

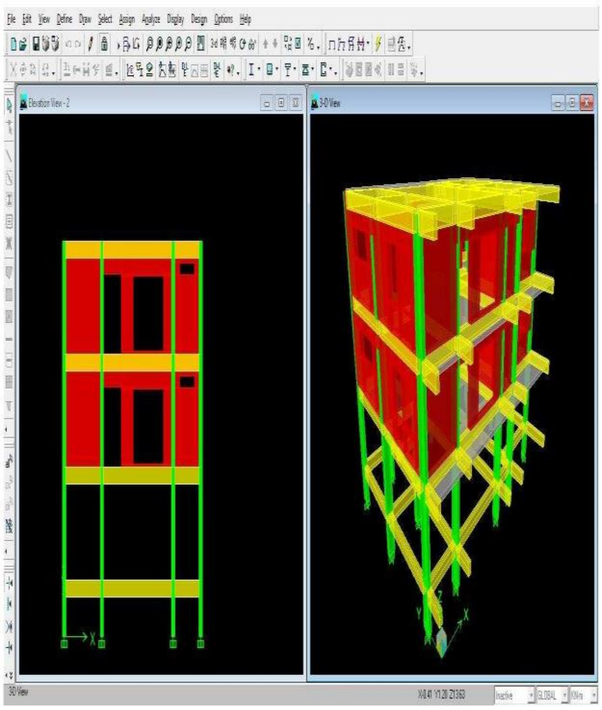


Figure 9a RC Frame With Ferrocement Panel As Infill

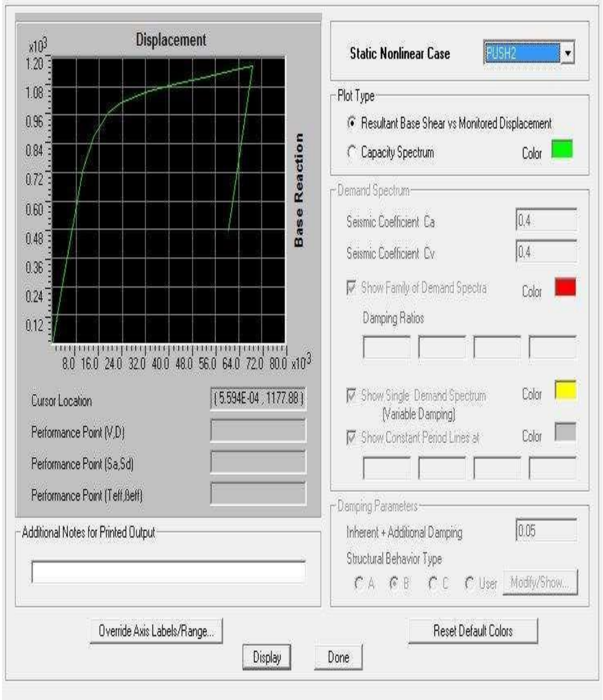


Figure 9b **Pushover Curve Obtained For Ferrocement Infilled Frame**

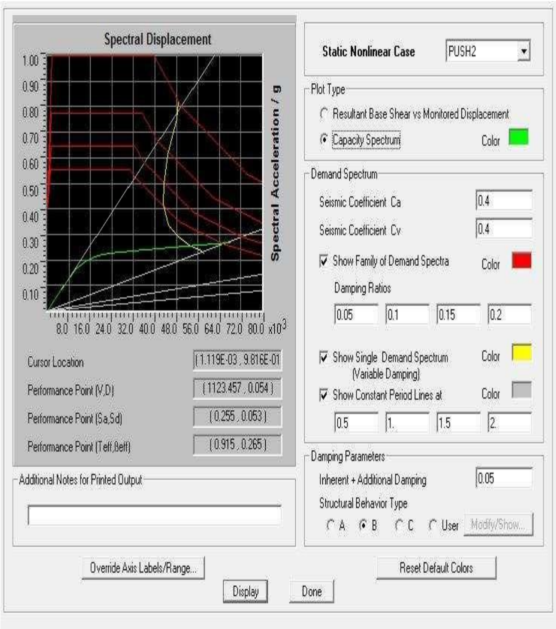


Figure 9c Capacity Demand Spectrum Obtained For FerrocementInfilled Frame

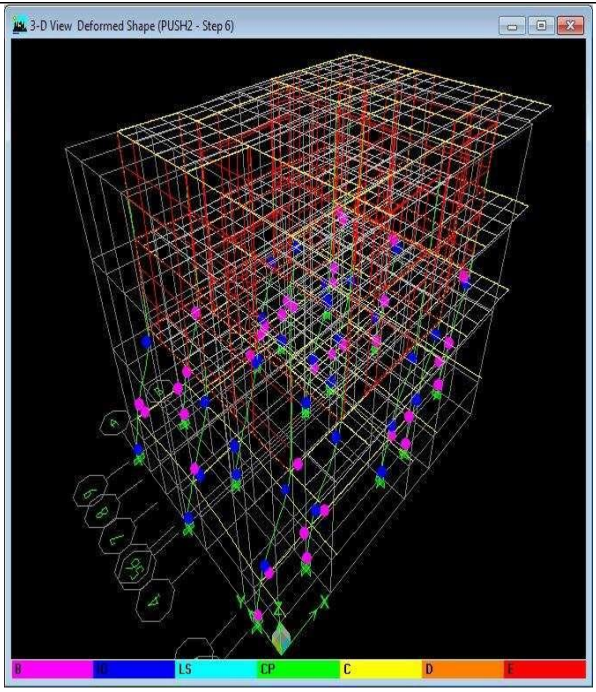


Figure 9d Hinge Levels Obtained For Ferrocement Infilled Frame

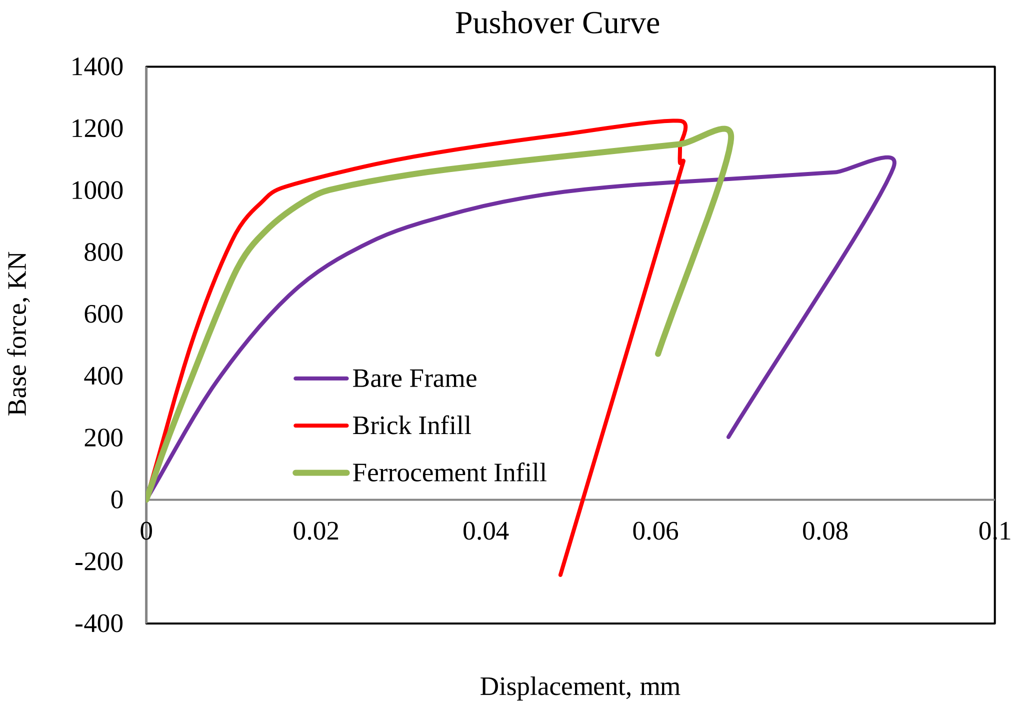


Figure 12 Combined Pushover curves for bare frame, Brick infilled and Ferrocement infilled frame

6.2 Discussions

Figure 12 presents the combined pushover curve for all the structural models Figure 7a, 7b, 7c and 7d represents the 3D model of bare frame, Pushoverv curve, capacity spectrum curve and hinge formations for bare frame model. Figure 8a, 8b, 8c and 8d represents the 3D model of bare frame, Pushoverv curve, capacity spectrum curve and hinge formations for brick infilled frame model. Figure 9a, 9b, 9c and 9d represents the 3D model of bare frame, Pushoverv curve, capacity spectrum curve and hinge formations for ferrocement cinfilled frame model. The following observations were made based on the obtained results.

1 RC bare frame reaches life safety level

2 RC frame infilled with brick undergoes IO-LS, that is the structure reaches life safety level s

3 RC frame infilled with ferrocement undergoes B-IO, that is the structure undergoes immediate occupancy level

4 The maximum base force of 1070.94KN and displacement of 0.088m for bare frame model.

5 For ferrocement infilled frame, the maximum base force of 1164.80KN and displacement of 0.0689m.

6 For brick infilled frame, the maximum base force of 1224.94KN and displacement of 0.0629m.

7 The structural effective time period with respect to spectral acceleration for bare frame is 0.765 and 0.304 respectively.

8 The structural effective time period with respect to spectral acceleration for brick infilled frame is 0.908 and 0.315 respectively.

9 The structural effective time period with respect to spectral acceleration for ferrocement infilled frame is 1.016 and 0.266 respectively.

6.3 Conclusions

1 The RC bare frame which is analysed for the static non linear pushover cases carries lower base force and gives maximum displacement.

2 When the same bare frame is infilled with ferrocement it proved that, ferrocement can take higher base force compared to bare frame and can offer more displacement compared to the brick infill.

3 Brick masonry can carry higher base shear but it cannot offer higher displacement when compared to ferrocement.

4 In the present study, RC Bare frame and brick infilled frame undergoes B-IO-IS i.e., linear to immediate occupancy level to life safety level ie structure have only minor damage in the structural component which are repairable.

5 Ferrocement infilled frame undergoes B-IO i.e., linear to immediate occupancy level i.e structure has only non structural damage which are repairable.

6 During earthquake, buildings are subjected to lateral displacement, ferrocement offers more displacement and less base force. The base force mainly depends on the mass of the structure by using ferrocement as infill mass of the structure can be reduced.

REFERENCES

[1]. Applied Technology Council, ATC-40 (1996), “Seismic Evaluation and Retrofit ofConcrete Buildings”, Applied Technology Council, Redwood City, California.

[2]. Ashraf Habibullah .S.E. (1998) “Practical three dimensional nonlinear static pushoveranalysis”, Structural Magazine, winter.

[3]. Armin B. Mehrabi and P. Benson Shing (1997)-Behaviour and analysis of masonry- infilled frames, ASCE Journal of Structural Engineering, Vol. 4 (March 1996), Issue 3,pp 320-331.

[4]. FEMA 273, (1997) “NEHRP Guidelines for the Seismic rehabilitation of Buildings,Federal Emergency Management Agency”, Washington D.C

[5]. Helmut Krawinkler and senevirantna G.D.PK. (1998).“ pros and cons of a pushover analysis of seismic performance evaluation” engineering structures, vol20, no-4-6,pp.452-464

[6]. Mehmet Inel, Hayri Baytan Ozmen. (2006). "Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings", Engineering Structures 28 1494–15
[7]. V S R Pavan kumar, Rayaprolu,P.Poluraju "Non Linear static analysis of multistoried building'' dept of civil engg,K L university.
[8]. Low Cost Earthquake Resistant Ferrocement Small House-M A Saleem(Jan 2008)
[9]. Vatwong Greepala and Pichai Nimityongskul* Influence of Heating Envelope on Post-Fire Mechanical Properties of Ferrocement Jackets
[10]. D. Raj Kumar and B. Vidivelli dept of civil and structural engg Acrylic rubber latex in ferrocment for strengthening reinforced concrete beams.
[11]. Unnikrishna Pillai S and Devdas Menon., "Reinforced concrete design" third edition, Tata McGraw- hill Education Private Limited. New Delhi.
[12]. Pankaj Agarwal and Manish shirkhande "Earthquake resistant design of structures" PHI Learning private limited New Delhi-1(2008)

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