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EFFECT OF PLASTIC HINGE PROPERTIES IN PUSHOVER ANALYSIS OF RC BUILDINGS

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CHAPTER 1

INTRODUCTION

Reinforced solid structures have been generally developed for business, mechanical and multi storied utilizations in seismic prone regions around the world. Structures fundamentally offer huge twisting, under a solid seismic tremor, a dynamic characteristic of the structure fluctuates, and exploring the execution of a structure requires inelastic methods representing these features.

Non-linear static has been created in the path of recent years and has became the dependable analytical system for outline and seismic overall performance assessment purposes. As pushover evaluation is normally applied for configuration and seismic execution assessment purposes, its regulations, shortcoming and the precision of its forecasts in routine utility have to be identified by using taking into account the factors influencing the pushover predictions. Plastic hinge is the one of the property which impacts the pushover analysis.

A plastic hinge in structural engineering refers back to the deformation of part of a beam anywhere plastic bending takes place. Hinges implies that having no capability to resist second. As a result, a plastic hinge behaves like a not unusual hinge enabling free rotation.Stress in the plastic areas is constant, when the complete move section anytime in a structure gets to be plastic, moment opposing is possible with out excessive pressure and a plastic hinge has been created. All the more such hinges are required for a complete breakdown. Plastic hinges reach out along the short length of the bar. Real estimations of these lengths rely on upon cross segment and load appropriations. Once a plastic hinge has been created at any cross area the moment of resistance at the point will stay constant until the breakdown of the entire structure has produced results. During strong earthquakes, reinforced concrete columns developed plastic deformations in regions, which after defined as plastic hinge regions.

Different researchers have given the different formulas to calculate the plastic hinge length. In the present study an RC building is analyzed by varying plastic hinge length and its location for bare and in filled frames.

1.1 SCOPE AND OBJECTIVE

1.1.1 Aim of the study:

The aim of the study is to evaluate the seismic performance of multi-storey buildings by varying the properties of plastic hinges.

The objectives of bare frame structure, and infilled structures, with possible uncertainties in various the present project is the pushover analysis of a G+4 RC structure considering it as hinge parameters.

1.1.2 Objectives:

- To obtain the response of buildings with and without infill at ground storey.
- Obtaining the response of buildings model by varying the properties of plastic hinge length suggested by various researchers.
- To compare the results obtained by varying the plastic hinge properties with the seismic performance of conventional multi-storey buildings.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

The purpose of the analysis is to evaluate the seismic safety of the structure. The review of this kind of seismic analysis is generally carried out for understanding the seismic resistance of building, past earthquake history damages to the buildings and construction practice adopted, type of building taken for study and its quantitative and qualitative aspects. Following are some of the studies of seismic analysis are taken for the present study.

2.2 REVIEW

Mehmet Inel (2006), Studied on the plastic hinge properties in non-linear analysis of framed buildings. They studied the feasible versions within the consequences of pushover analysis due to default and individual referred to detail homes they considered 4 and 7 storey houses to represent low and medium upward push buildings for his or her advantage know-how of. Plastic hinge period and transvers reinforcement spacing are assumed to be brilliant parameters in the character defined hinge houses, observations surely indicates that the character described hinge version is fine than the default hinge model in reflecting nonlinear habits well matched with the detail residences. However, if the default-hinge mannequin is preferred due to simplicity, the customer should be aware about what is provided within the software and will need to save you the misuse of default-hinge houses.

Rajesh P. Dhakal and Richard C. Fenwick (2008), Studied on the detailing of plastic hinges in seismic design of concrete structures. They explained why the structural ductility factor doesn't give a secure guidance to the deformation sustained in an man or woman plastic hinge, located on take a look at outcome of 37 beams, 25 columns and 36 walls. The layout curvature limits are proposed for one in every of a type categories of plastic hinge.

Li Peng and Yi Weijian (2008), Based on the experiment analysis under cycle loading, the effect of different axial load ratio and loading path on columns seismic capability is compared. It has been found that axial load ratio and loading path affect plastic hinge length obviously. Plastic hinge length increased as axial load ratio increased. Before concrete crushed, transverse reinforcement provided confining stress both in tensile and compressive section. Strains of transverse reinforcement increased rapidly after it yielded.

R. Shreekala, N. Lakshmanan (2009), studied on aseismic design with predefined plastic hinges. They highlighted the importance of selection of suitable ductile composite materials, incorporating it into the predefined locations, for better seismic performance. Their idea is to replace the normal concrete with ductile composites at plastic hinge locations, to establish this concept they carried out simple experimental investigations.

Mehmet Alpaslan (2014), Studied on the Use of regression analysis in determining the period of plastic hinge in bolstered concrete columns. Basic goal of their study is to create a regressional analysis method that can estimate the period of the plastic hinge that's an important layout parameter in pushover analysis. They did experimental studies on reinforced square concrete columns and they collected the test results of 170 different square reinforced concrete column from the existing literature,

Ravikumar H S, Supriya R Kulkarni (2015), studied on the plastic hinge formation on reinforced concrete frame by non-linear static analysis. They evaluated the expected overall performance of a structure through estimating its pressure and deformation needs to layout ground motions by means of the use of static inelastic evaluation. In this evaluation the motives regarded are worldwide go along with the glide, interstorey drifts, inelastic aspect deformation, deformation among elements and lots of others., reasons which ends up on the extremely good deformation potential moreover and it is based at the extremely good curvature and plastic hinge homes. They made an try and recognize the order of hinge formation for floor motions and thereby tried to decorate the accuracy of pushover evaluation. They did analysis on single storied strengthened concrete body through using the application package SAP 2000

2.3 SUMMARY

The literature survey gives the plastic hinge properties in non-linear analysis of framed buildings. To the extent analysis is carried out by considering the different plastic hinge length suggested by various researchers.

CHAPTER 3

EARHQUAKE ANALYSIS AND DESIGN OF STRUCTURE

3.1 Seismic Design Philosophy

The philosophy of seismic design can be summarized as follows

i) Amid the minor yet visit ground movements, the auxiliary individuals from the building that convey vertical and level powers ought not be harmed; however non-basic individuals may maintain repairable harm.

ii) Amid moderate yet intermittent ground movements, the auxiliary individuals may support repairable harm, while the other non-basic individuals from the building might be harmed such that they may be supplanted after the quake.

iii) Amid solid but ground movements, the auxiliary individuals may maintain extreme harm, yet the building ought not to crumple.

3.2 DETERMINATION OF LATERAL DESIGN FORCES

It is perceived from outline theory that the complete security against seismic tremor of all sizes is not financially plausible and plan construct alone with respect to quality criteria is not legitimized. The essential configuration criteria of quake safe outline ought to be founded on sidelong quality and also deformability and pliability limit of structure with constrained harm however not crumple. In progression for a working to fulfill the above configuration theory, it is of most extreme significance that the outline burdens be assessed with adequate level of exactness. Significant deviations from the genuine seismic burdens will bring about the building being either perilous or uneconomical. Notwithstanding assessing the outline stacks accurately, the enumerating ought to guarantee that the building has adequate flexibility keeping in mind the end goal to fulfill the third necessity of the configuration rationality. In this manner seismic safe configuration of structures requires the correct estimation of the design loads and proper detailing.

The techniques to decide parallel powers in the code, IS 1893 (Part 1): 2002 depend on the guess impacts, yielding can be represented direct investigation of the building utilizing the outline range. This investigation is completed either by modular examination strategy or element examination system. An improved strategy may likewise be embraced that will be alluded as parallel power technique or comparable static method. The primary distinction between the identical static technique and element investigation system lies in the greatness and conveyance of sidelong powers over the tallness of the structures. In the dynamic examination strategy the parallel powers depend on properties of the regular vibration methods of the building, which are controlled by circulation of mass and solidness over stature. In the identical parallel power method the size of strengths depends on an estimation of the key time frame and on the conveyance of powers as given by a basic recipe that is fitting just for standard structures. The accompanying areas will examine in subtle element the aforementioned proportional static and the dynamic technique to decide the outline parallel strengths in point of interest.

3.3 EQUIVALENT STATIC METHOD

Following procedure is generally used for the equivalent static analysis:

i) Determination of base shear (V_B) of the building

$$V_B = A_h \times W$$

Where,

A_h =Design horizontal seismic coefficient for a structure

W = seismic weight of building

A_h shall be determined by the following expression:

$$A_h = \frac{Z \cdot I \cdot S_a}{2 \cdot R \cdot g}$$

Provided that for any structure with T ≤ 0.1s, the value of A_h will not be taken less than Z/2 whatever be the value of (I/R).

Where,

Z = Zone factor given in Table 2 of IS 1893 (Part 1): 2002, is for the Maximum Considered Earthquake (MCE) and service life of structure in a zone. The factor 2 in the denominator of Z is used so as to reduce the maximum considered earthquake zone factor for Design Basic earthquake.

I = Importance factor, depending upon the functional use of the structures, characterized by hazardous consequences of its failure, Post earthquake functional needs, Historical value, or economics importance (Table -6 of IS 1893 (Part 1): 2002).

R =Response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. However the ratio of (I/R) shall not be greater than 1.0 .The values of R for buildings are given in Table 7 of IS 1893 (Part 1): 2002.

Sa/g = Average response acceleration coefficient for rock or soil sites as given in fig 2 and Table 2 of IS 1893 (Part 1): 2002.

The included reduction factor "*R*" in base shear formula is an attempt to consider the structures inelastic characteristics in linear analysis method, since it is undesirable as well as uneconomical that a structure will be designed on the basis that it will remain in elastic range for all major earthquakes. The response reduction factor *R* is also called response modification factor or behavior factor. Seismic weight of building (*w*) is the sum of the seismic weight of floors; the seismic weight at any floor level would be equal to dead weight of the floor system plus weight of column and walls.

i) Lateral distribution of design base shear:

The design base shear V_B thus obtained is then distributed along the height of the building using a parabolic distribution expression:

$$Q_i = V_B \frac{W_i \cdot h_i^2}{\sum_{i=1}^{n} W_i \cdot h_i^2}$$

Where

Q_i = Design lateral force at i^{th} floor

W = Seismic weight of i^{th} floor

h_i = Height of floor measured from base

n = Number of stories in the building is the number of levels at which masses are located.

3.4 PUSHOVER ANALYSIS

A pushover evaluation is performed via way of subjecting a structure to a monotonically increasing instance of horizontal burdens, talking to the inertial powers which would be professional with the aid of using the structure whilst subjected to floor shaking. Underneath incrementally increasing masses special auxiliary accessories might also yield consecutively. Utilizing a weakling examination, a trademark nonlinear vigor relocation relationship can also be resolved. It is critical for the following concerns:

- Pushover analysis is a nonlinear static evaluation used as a rule for seismic evaluation of framed constructing.

- Seismic needs are computed by means of nonlinear static evaluation of the structure, which is subjected to monotonically increasing lateral forces with an invariant peak-wise distribution until a goal displacement is reached.

- it is also vital fork evaluating the seismic adequacy of current constructions.

3.4.1 Standard Pushover Method:

The The pushover assessment includes the usage of gravity loads and consultant lateral load sample. The lateral lots have been implemented monotonically in a regulated1 nonlinear static research. Associated lateral loads had been growing velocities within the x bearing talking to the forces which may be skilled through way of the structures whilst subjected to ground shaking. A predefined lateral load layout that is dispersed along the constructing immoderate is then implemented. The lateral people are extended except some of members yield. The capability of the shape is represented thru the lowest shear rather than roof- displacement graph as proven in confirm.

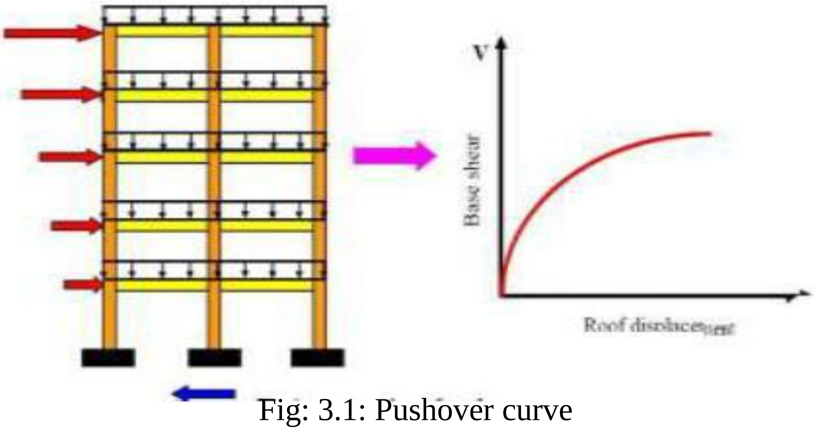
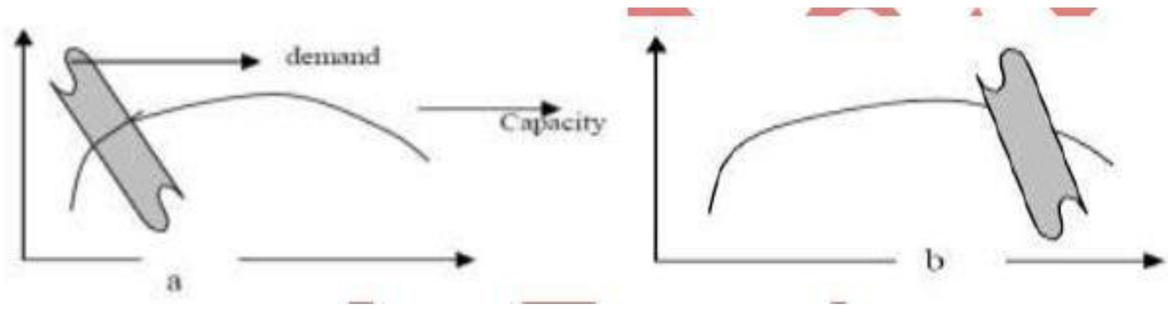


Fig: 3.1: Pushover curve



(a) Safe Design (b) Unsafe Design

The maximum vital output of a pushover assessment is in terms of reaction demand versus capability. If the demand curve intersects the capability envelope close the elastic range, Fig (a), then the shape has a simply proper resistance. If the demand curve intersects the capability curve with little reserve of energy and deformation potential, Fig (b), then it may be concluded that the shape will behave poorly during the imposed seismic excitation and need to be retrofitted to keep away from destiny essential damage or deliver manner

Underneath incrementally increasing hundreds some elements may yield sequentially. For this reason, at each event, the structures experiences a stiffness change as shown in figure 3.2, where IO,LS and CP stand for instant occupancy, life safeguard and collapse1 prevention respectively.

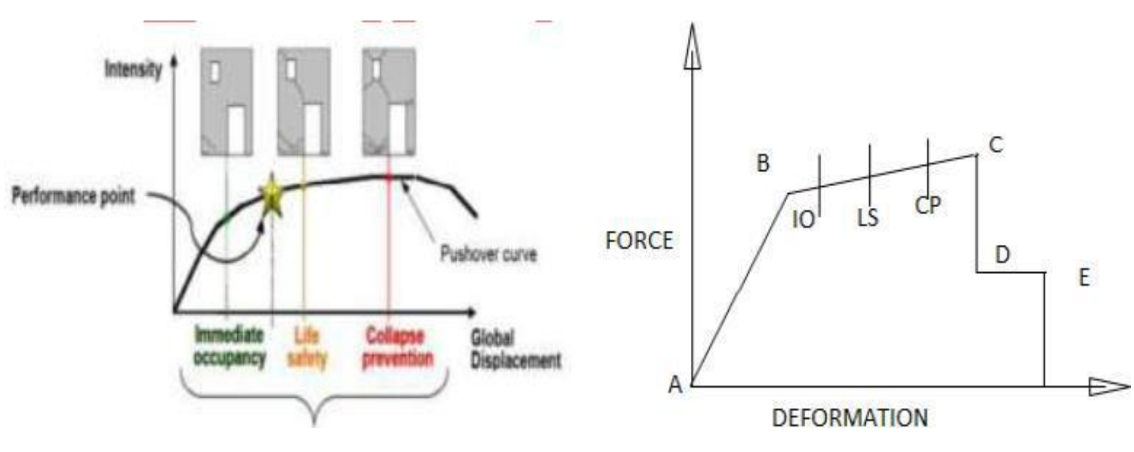


Fig: 3.2: Performance Level Described by Pushover Analysis

Immediate occupancy IO: damage is relatively limited; the structure retains a significant portion of its original stiffness.

Life safety level LS: substantial damage has occurred to the structure, and it may have lost a significant amount of its original stiffness. However, a substantial margin remains for additional lateral deformation before collapse would occur.

Collapse prevention CP: At this level the building has experienced extreme damage, if laterally deformed beyond this point; the structure can experience instability and collapse.

3.5 PURPOSE OF PUSHOVER ANALYSIS

The reason for pushover evaluation is to evaluate the ordinary execution of structural systems with the aid of comparing execution of a structural device via estimating its strength and deformation needs in define of quakes by using approach for static inelastic research and contrasting these requests with reachable limits at the execution tiers of top class. The assessment depends on an appraisal of essential execution parameters, along with international float; inter storey go with the flow, inelastic aspect detail deformation among factors.

The advantages of pushover analysis are as follows.

1. Dynamic evaluation although accurate takes very long term. NSP assessment on special hand takes only a fraction of time to present priceless outcome. On the grounds that point is very important parameter in layout area approximate outcome can also be efficaciously implemented to derive a precious conclusion. This makes NSP evaluation manner greater relevant in design place of job.
2. Analytically sold ability curve suggests the premature failure and weak spot of the structure.
3. The plastic hinge formation, stiffness degradation, cave in load and ductility of the structure will also be monitored.

3.6 METHOD TO PERFORM PUSHOVER ANALYSIS

The two key purposes of an execution based totally define system are demand and capacity. Demand is the illustration of the seismic tremor ground movement. Capacity is illustration of the structure's capacity to oppose the seismic call for. The performance is difficulty to the manner that the limit can cope with the interest. As such, the shape should have the ability to oppose the demand of the seismic tremor such that the execution of the structure is right with the goals of the configuration. Rearranged nonlinear investigation method using pushover techniques, require willpower of three vital additives.

a) Capacity

b) Demand (displacement)

c) Performance.

3.7 STEP-BY-STEP METHOD TO DETERMINE CAPACITY

Pushover analysis is a device to get the load displacement plot for a structure or a member. It includes applying loads to the model of the structure incrementally, i.e., pushing the structure, and plotting the total applied force and the associated displacement at every augmentation. With the expansion in the magnitude of loading, the powerless connections and disappointment methods of the structure are found.

The following procedure can be used to obtain a pushover curve.

1. Create a 3D model for the Building.
2. Assign rigid zone factor as 1 for beam-column joint end-offset, which specifies that the fraction of each end-offset assumed to be rigid for bending and shear deformation.
3. Assign hinge properties available in SAP Nonlinear as per ATC-40 to the frame elements. For the beam default hinge that yields based upon the flexure (M3) is assigned, for the column default hinge that yields based upon the interaction of the axial force and bending moment (P M2 M3) is assigned, and for the equivalent diagonal strut default hinge that yields based upon the axial force (P) only is assigned.
4. Define the load combinations. The analysis should also include gravity loads. Define the load cases to perform the static linear analysis.

5 .After the linear static analysis, design of the building as per IS-456 2000, is performed for the defined load combinations, so that the hinge properties are generated for the assigned frame elements.

6. After the design of the building, the static pushover analysis is carried out to establish the performance point.

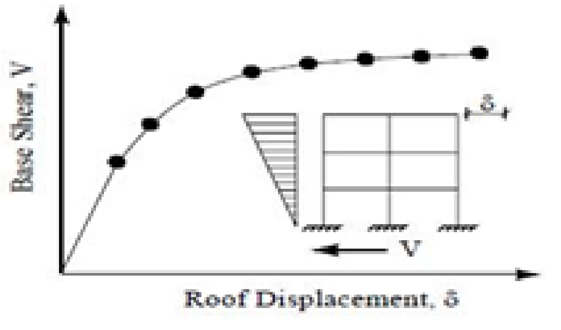


Figure: 3.3: Generation of Pushover curve

CHAPTER 4

MODELING AND ANALYSIS OF THE STRUCTURE

4.1 GEOMETRY

The structure is a Ground + four storied RC bare framed structure. Following figures shows the basic overall geometry of the structure.

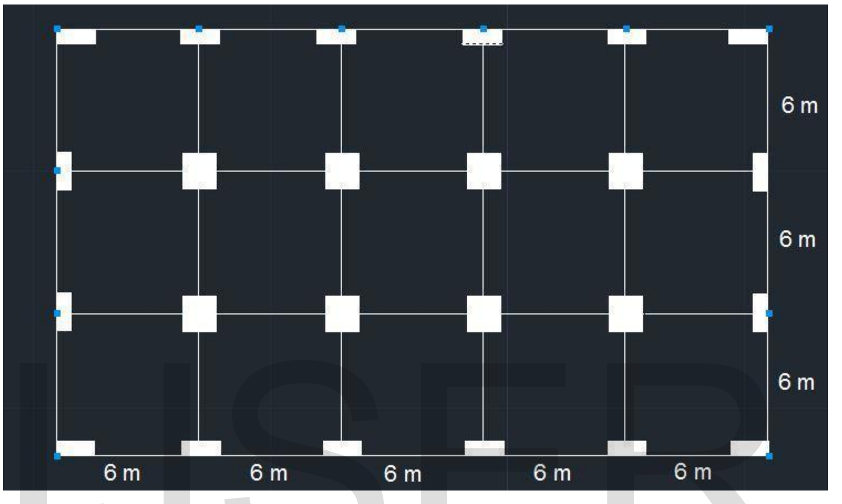


Fig: 4.1.1: Plan of Building

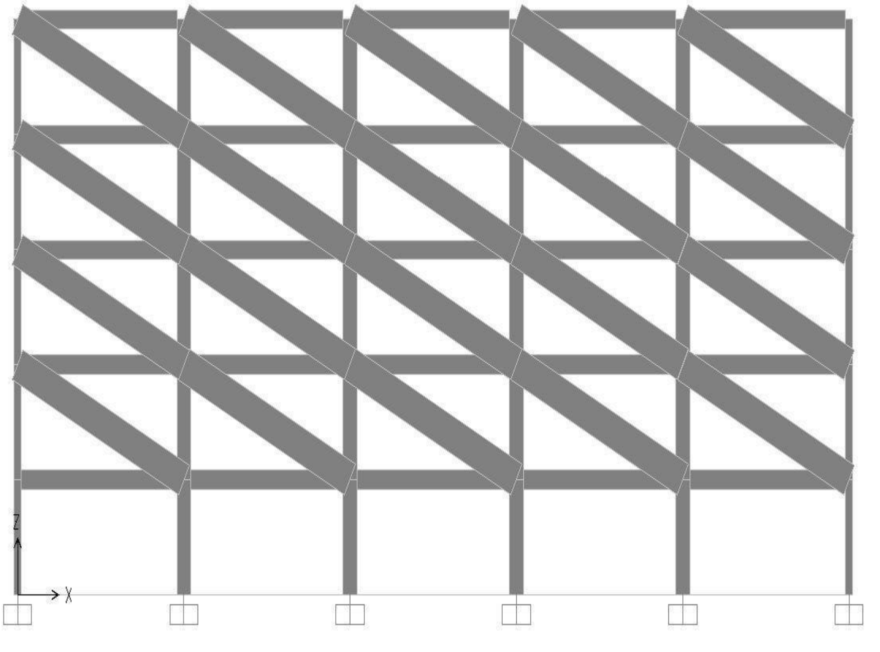


Fig: 4.1.2: Elevation of infill frame

4.2 SECTION PROPERTIES

Beam sections are 250mm x 500mm in size with 16Φ reinforcement bars at top and bottom in beam. External columns are of size 250mm x 500mm and internal columns are of size 500mm x 500mm with 16Φ reinforcement bars at top and bottom in columns. The transverse reinforcement for both beams and columns is provided 8Φ stirrups/ties. The slab is 100 mm thick.

4.3 MATERIAL PROPERTIES

The design material properties for the structure:

Concrete Grade: M30

Reinforcement Grade: HYSD (Fe415)

4.4 LOAD COMBINATIONS

The following load combinations are considered for the analysis and design as per IS: 1893-2002.

Table: 4.1: Load combinations as per IS: 1893-2002 and IS: 875(Part3)-1987

Load Combination	Load Factors
Gravity analysis	1.5 (DL+LL)
Equivalent static Analysis	1.2 (DL+ LL ± EQ_X) 1.5 (DL± EQ_X) 0.9(DL ± EQ_X)

4.5 PLASTIC HINGE LENGTH

A plastic hinge in structural engineering refers to the deformation of a part of a beam anywhere plastic bending happens. Hinge means that having no capacity to withstand moment. Therefore, a plastic hinge behaves like a trendy hinge allowing free rotation.

4.6 PLASTIC HINGE LENGTH FORMULATIONS

Various empirical expressions have been proposed by investigators for the equivalent length of plastic hinge $\boldsymbol{l_p}$.

1. **Sawyer's formula**

   Lp = 0.25d+0.075L

2. **Mattock's formula**

   Lp = 0.5d+0.05L

3. **Priestley-Park's formula**

   Lp = 0.08L+db

4. **Paulay-Priestley' formula**

   Lp = 0.08L+0.022dbfsy

5. **Berry's formula**

   Lp = 0.05L+((0.01dbfsy)/(sqrt(fck)))

Where,

Lp = Plastic hinge length

d = Effective depth of the member

db = Diameter of longitudinal reinforcement

fsy = Yield strength of reinforcement bars, in MPa.

fck = Compressive strength of concrete

A large variation may be noted in the plastic hinge lengths calculated by different formulations given above.

Table: 4.2: Calculation of plastic hinge length using above formulae for beam and column

Researchers	Formulation	Plastic hinge length	
		Beam	Column
Sawyer	Lp = 0.25d+0.075L	0.58	0.35
Mattock	Lp = 0.5d+0.05L	0.55	0.40
Priestley & Park	Lp = 0.08L+db	0.48	0.24
Paulay & Priestley	Lp = 0.08L+0.022dbfsy	0.63	0.39
Berry	Lp=0.05L+((0.01dbfsy)/(sqrt(fc)))	0.42	0.27

4.7 ANALYSIS IN SAP 2000

The following steps are included in the pushover analysis for modeling.

Step 1: Create the basic computer model (without the pushover data) using the graphical interface of SAP 2000 and define the material properties, geometric properties.

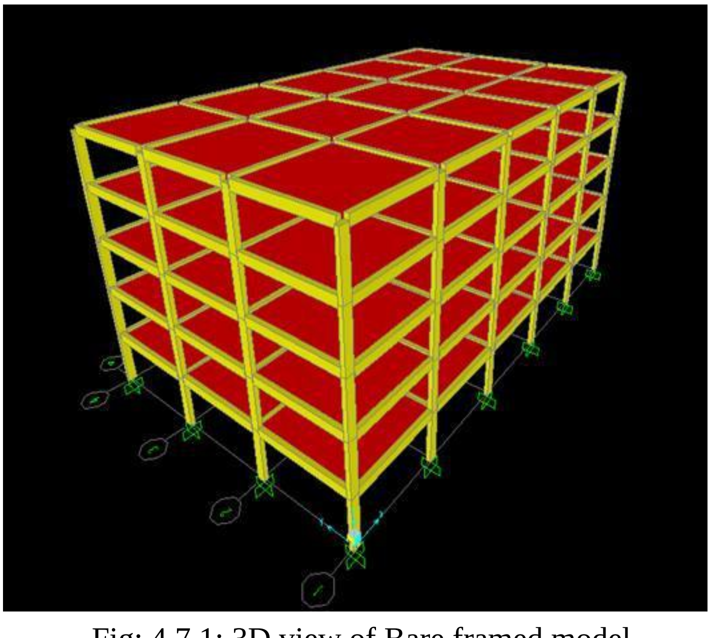


Fig: 4.7.1: 3D view of Bare framed model

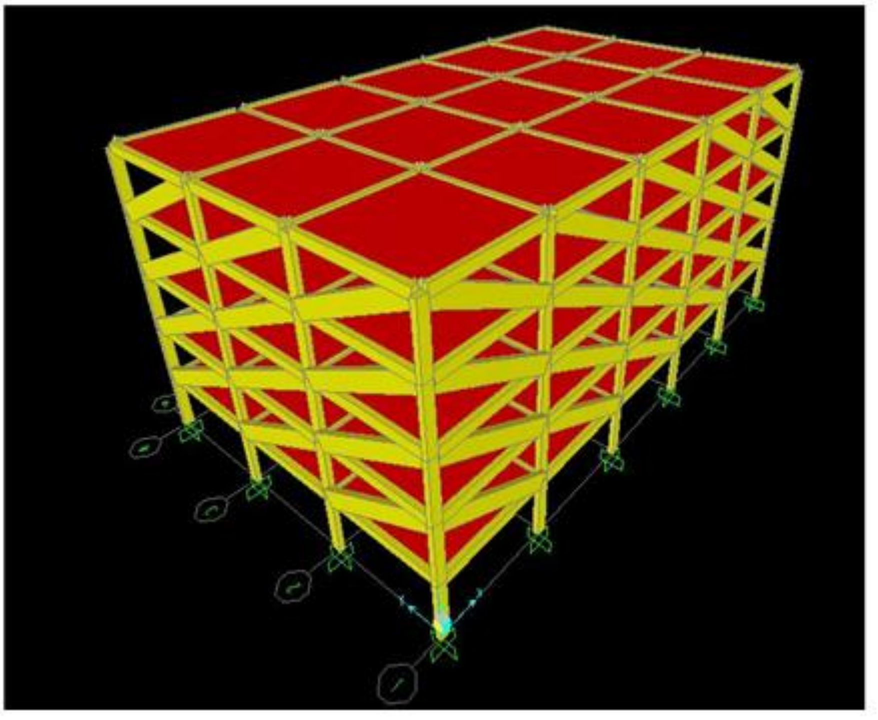


Fig: 4.7.2: 3D view of in filled framed model

Step 2: The user defined moment-curvature data was entered to define the hinge properties based on the material nonlinearity. M3 plastic hinges (user-defined) was assigned to beam and P(M2 M3) plastic hinges assigned to the columns at both ends of the beams and columns, to incorporate elemental nonlinearity

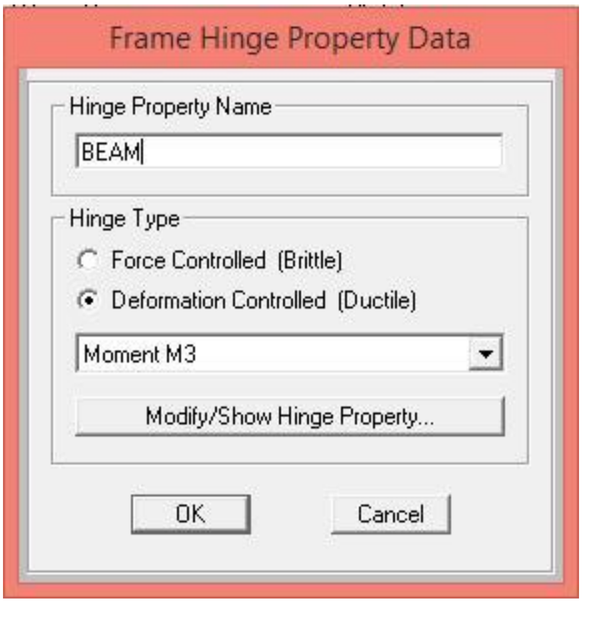


Fig: 4.7.3: Frame hinge property data

Step 3: Define the pushover load instances. Typically the first pushover load case is used to apply gravity load after which next lateral pushover load instances are special to begin from the final situations of the gravity pushover.

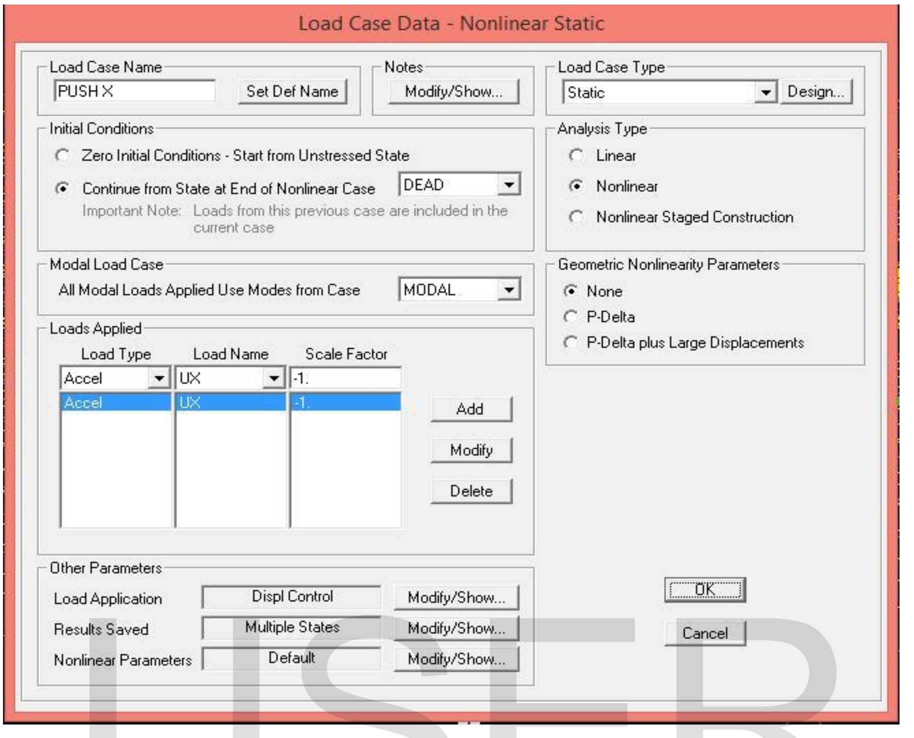


Fig: 4.7.4: Giving static nonlinear analysis command

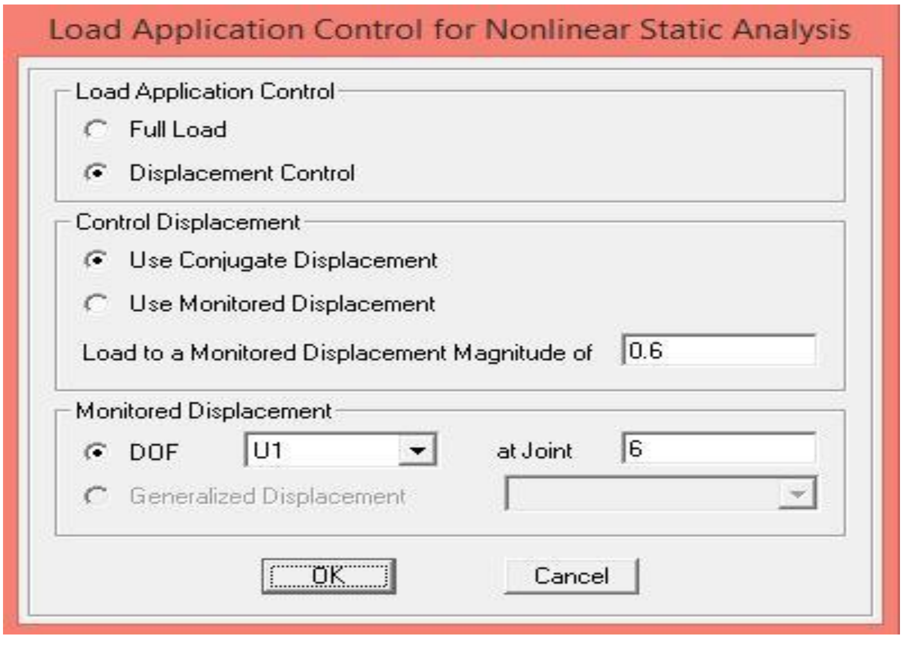


Fig: 4.7.5: Details of displacement controlled analysis assignment

Step 4: Define the evaluation case. Nonlinear static pushover analysis, displacement controlled turned into defined for the prevailing look at.

Step 5: Run the nonlinear static pushover analysis for the above model.

Step 6: Obtain the pushover curve.

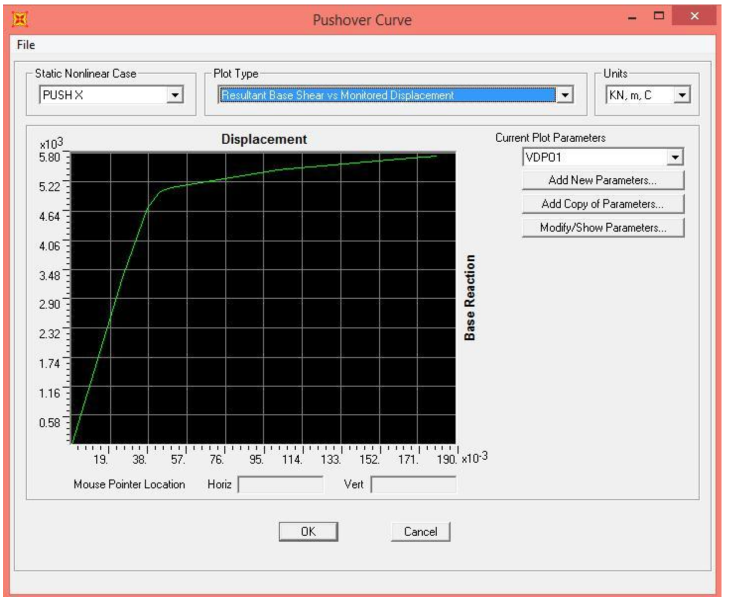


Fig: 4.7.6: Pushover Curve

CHAPTER 5

ANALYSIS RESULTS

The analytically obtained pushover curves for different models using SAP 2000 are shown below.

5.1 Bare frame models with hinges assigned using different hinge length formulations

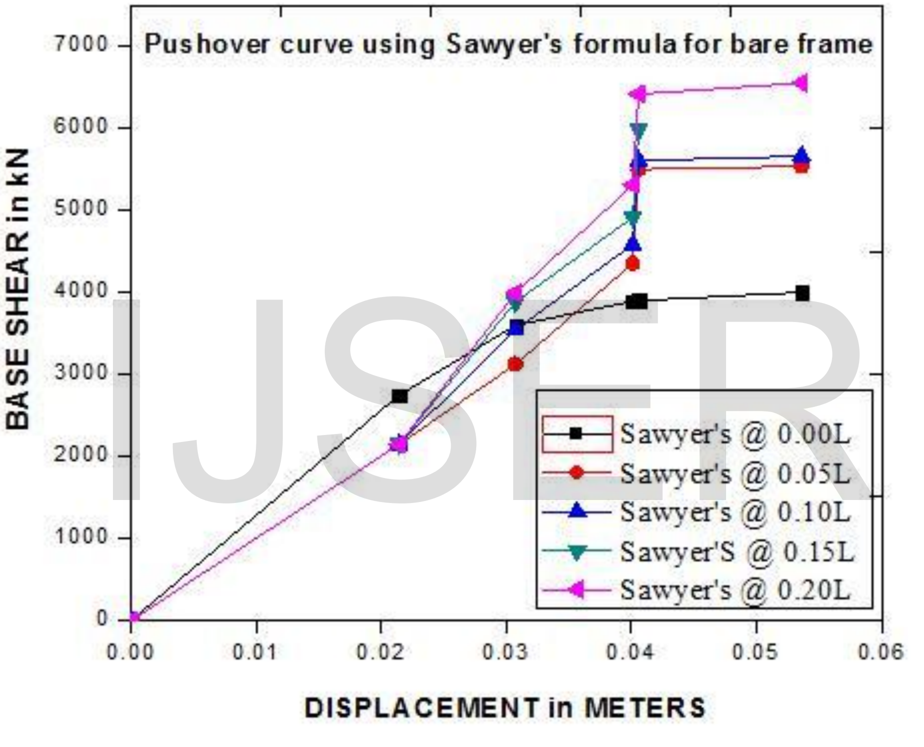


Fig: 5.1.1: Pushover curves using Sawyer's formula for bare frame

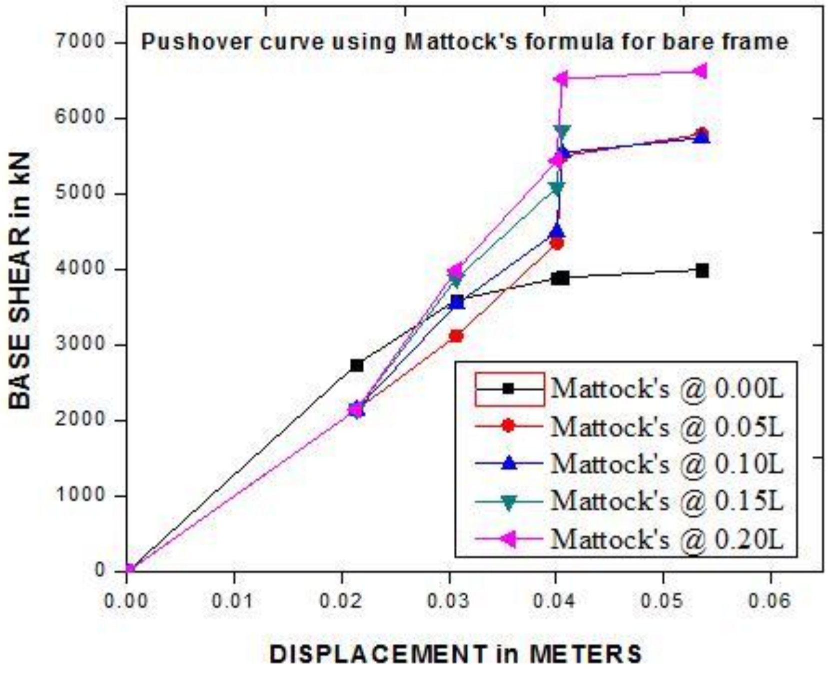


Fig: 5.1.2: Pushover curves using Mattock's formula for bare frame

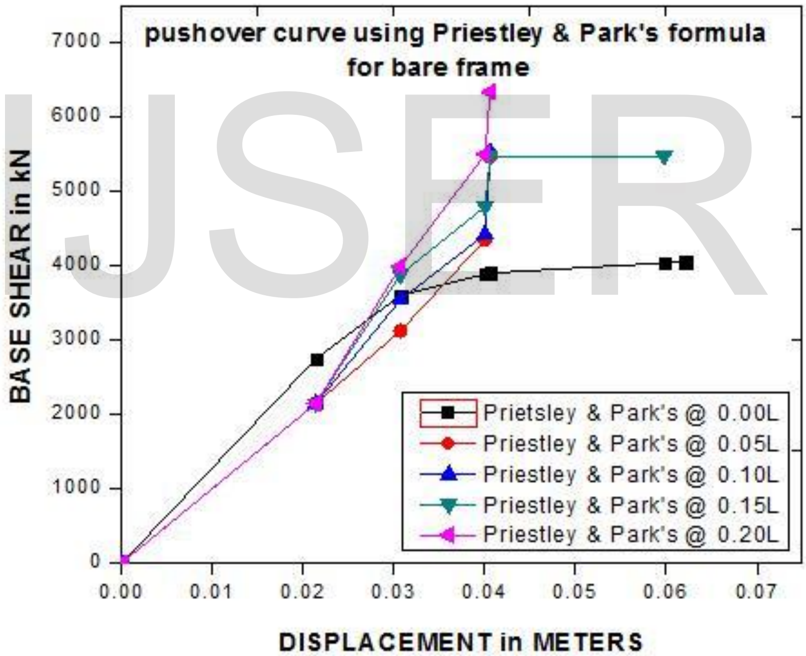


Fig: 5.1.3: Pushover curves using Priestley & Park's formula for bare frame

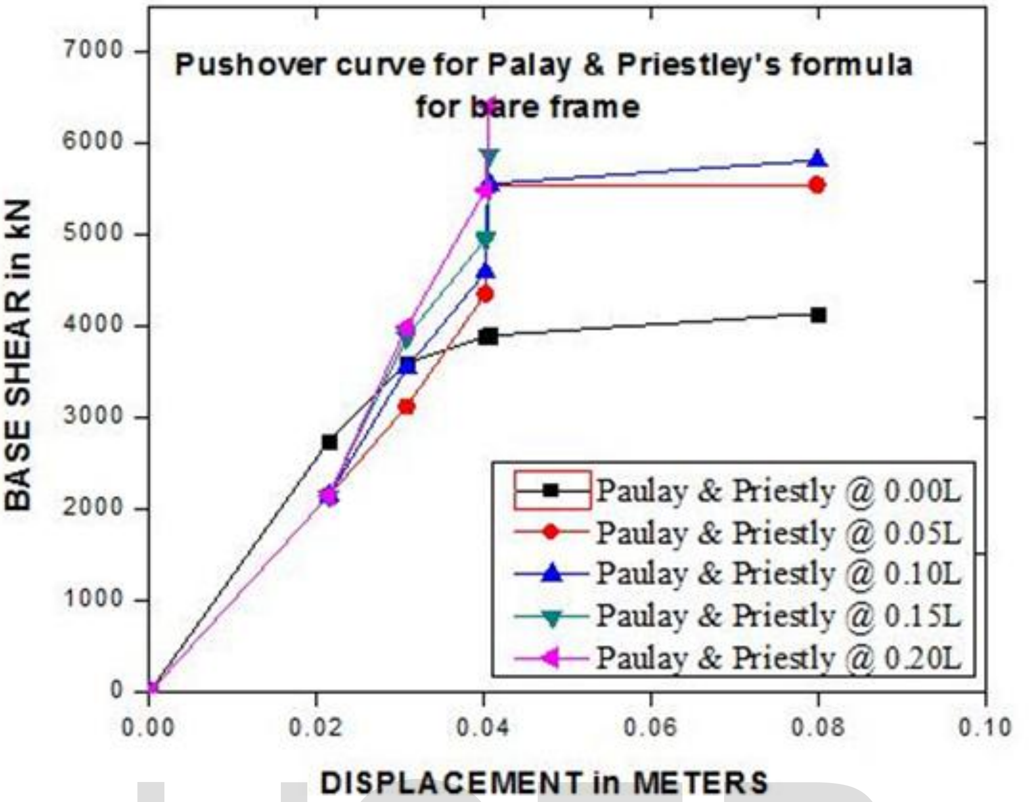


Fig: 5.1.4: Pushover curves using Paulay-Priestley's formula for bare frame

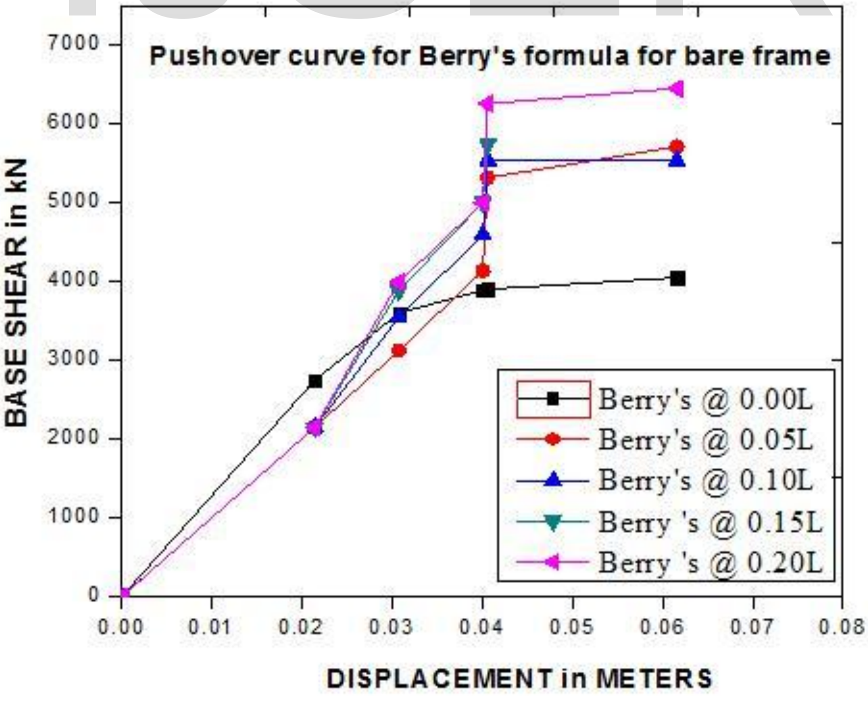


Fig: 5.1.5: Pushover curves using Berry's formula for bare frame

(Base Shear in kN and Displacement in meters)

Table: 5.1: Base shear & displacement values for bare frame

Hinge location from support / Formula	0.0L	0.05L	0.10L	0.15L	0.20L
Sawyer's	P =5295.106 Δ = 0.109	P =5337.330 Δ =0.108	P =5513.903 Δ =0.105	P =5655.171 Δ =0.104	P =5778.147 Δ =0.103
Mattock's	P =5295.106 Δ =0.109	P =5321.638 Δ =0.108	P =5523.496 Δ =0.105	P =5689.622 Δ =0.104	P =5817.650 Δ =0.102
Priestley & Park's	P =3054.968 Δ =0.025	P =3058.890 Δ =0.025	P =3060.415 Δ =0.026	P =3078.035 Δ =0.026	P =3088.892 Δ =0.026
Pauley-Priestley's	P =5294.827 Δ =0.109	P =5315.530 Δ =0.108	P =5575.122 Δ =0.105	P =5661.175 Δ =0.104	P =5818.163 Δ =0.103
Berry's	P =5304.071 Δ =0.109	P =5357.553 Δ =0.108	P =5511.384 Δ =0.105	P =5703.080 Δ =0.104	P =5792.974 Δ =0.103

5.2 Infilled frame models with hinges assigned using different hinge length formulations

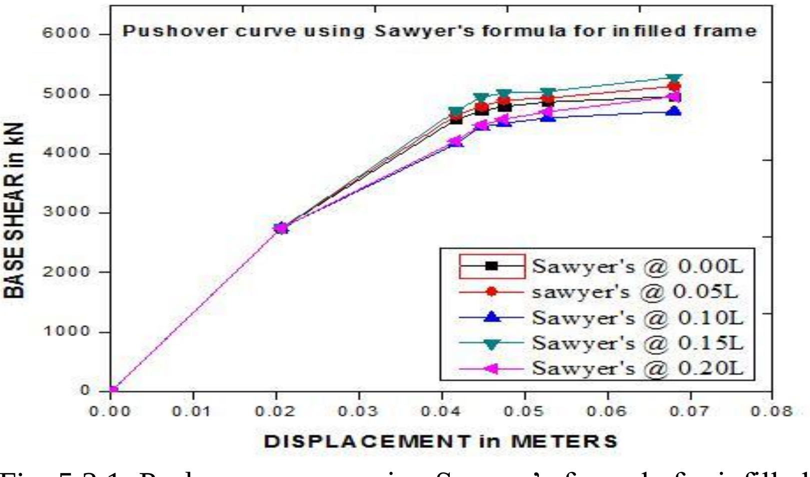


Fig: 5.2.1: Pushover curves using Sawyer's formula for infilled frame

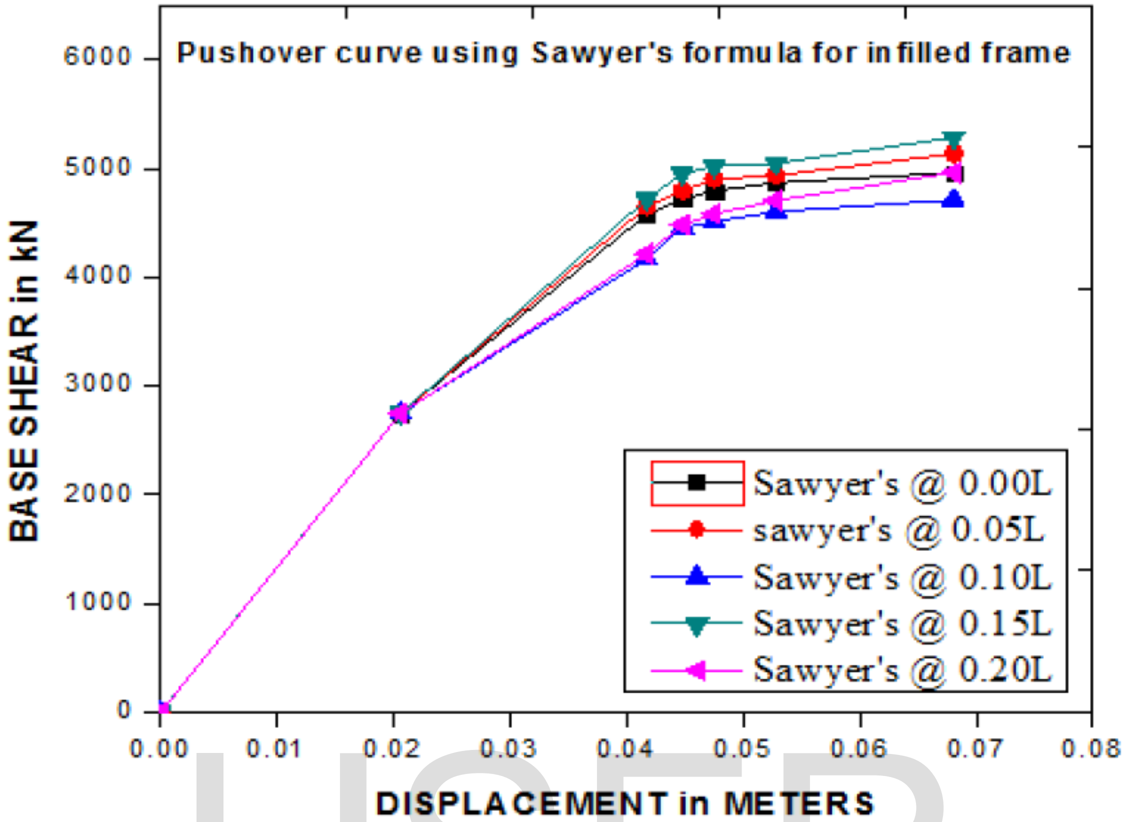


Fig: 5.2.2: Pushover curves using Mattock's formula for infilled frame

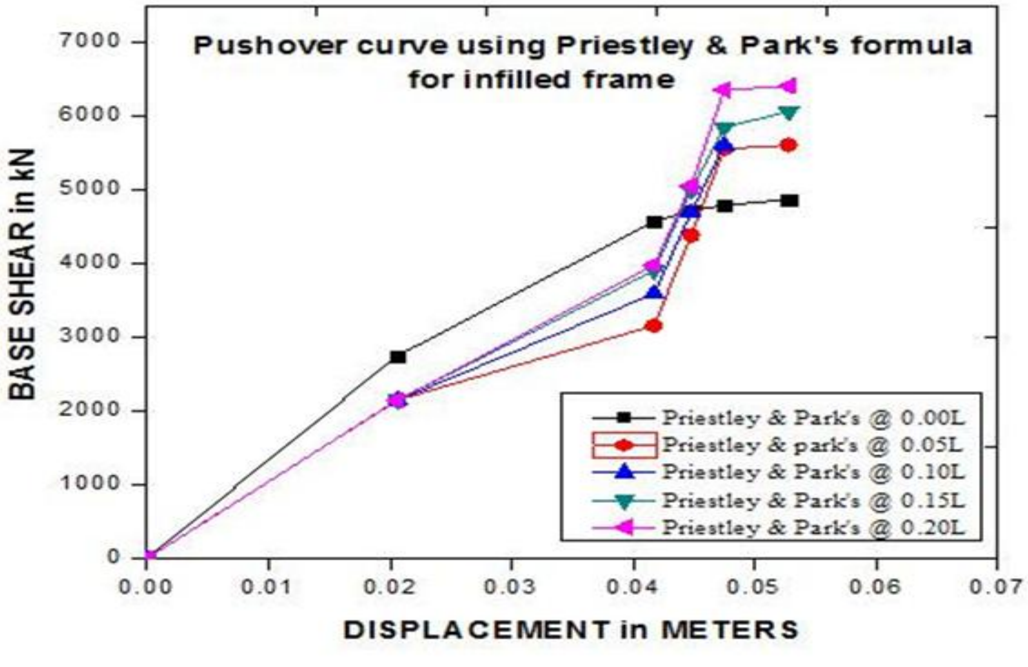


Fig: 5.2.3: Pushover curves using Priestley & Park's formula for infilled frame

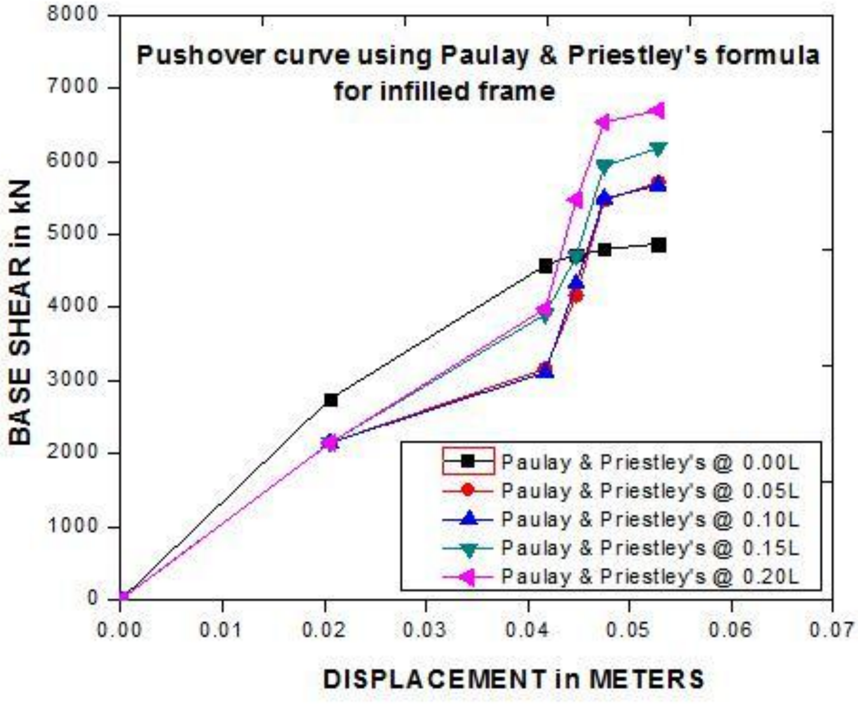


Fig: 5.2.4: Pushover curves using Paulay-Priestley's formula infilled for frame

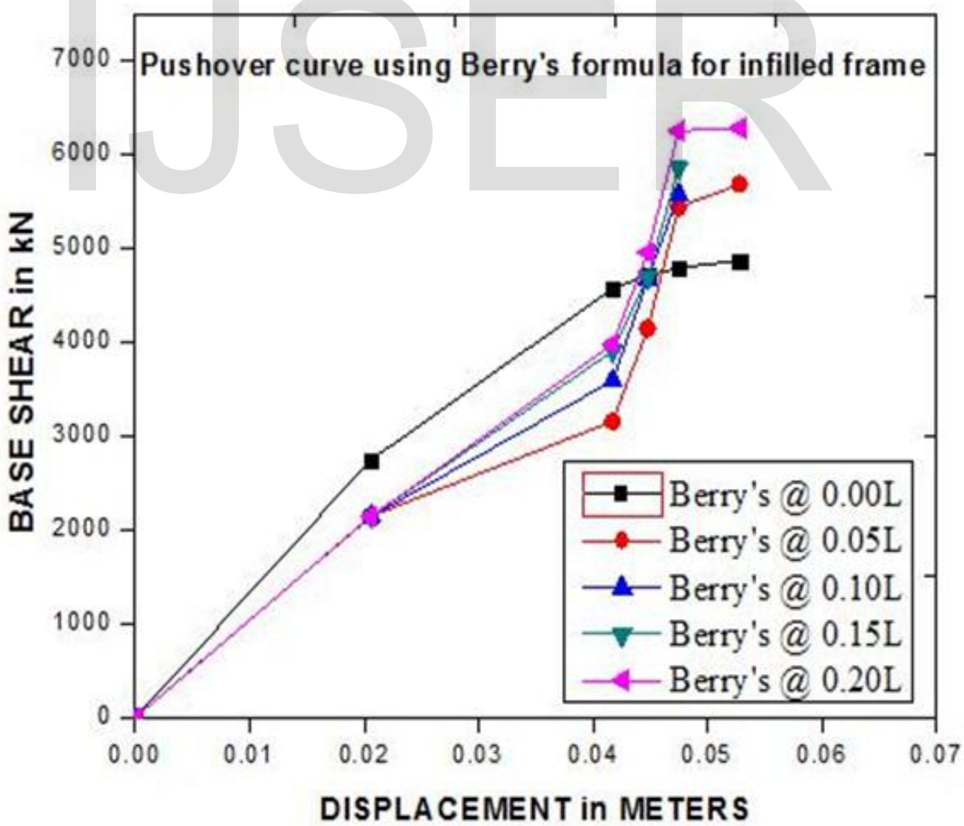


Fig: 5.2.5: Pushover curves using Berry's formula for infilled frame

(Base Shear in kN and Displacement in meters)

Table: 5.2: Base shear & displacement values for infilled frame

Hinge location from support / **Formula**	**0.0L**	**0.05L**	**0.10L**	**0.15L**	**0.20L**
Sawyer's	P =5182.752 Δ = 0.132	P =5258.494 Δ =0.130	P =5361.687 Δ =0.137	P =5497.955 Δ =0.128	P =5504.566 Δ =0.136
Mattock's	P =5188.060 Δ =0.132	P =5264.336 Δ =0.131	P =5366.876 Δ =0.137	P =5411.756 Δ =0.128	P =5515.832 Δ =0.136
Priestly&	P =5237.392	P =5487.195	P =5571.463	P =6033.760	P =6213.760
Park's	Δ =0.131	Δ =0.112	Δ =0.105	Δ =0.116	Δ =0.117
Paulay-Priestley's	P =5763.537 Δ =0.136	P =5769.047 Δ =0.124	P =5884.248 Δ =0.122	P =5972.970 Δ =0.118	P =6443.781 Δ =0.128
Berry's	P =5462.273 Δ =0.113	P =5495.949 Δ =0.112	P =5545.105 Δ =0.101	P =5848.243 Δ =0.107	P =5937.513 Δ =0.106

5.3 Comparison of pushover curves for bare frame with hinges of different hinge length formulations

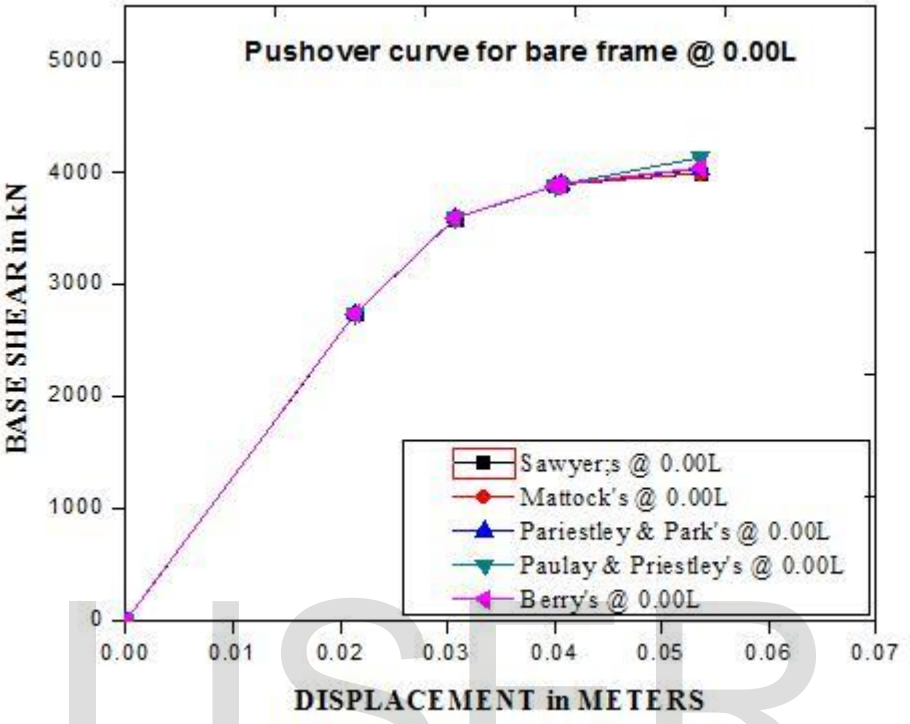


Fig: 5.3.1: Pushover curves for bare frame with hinges at 0.0L

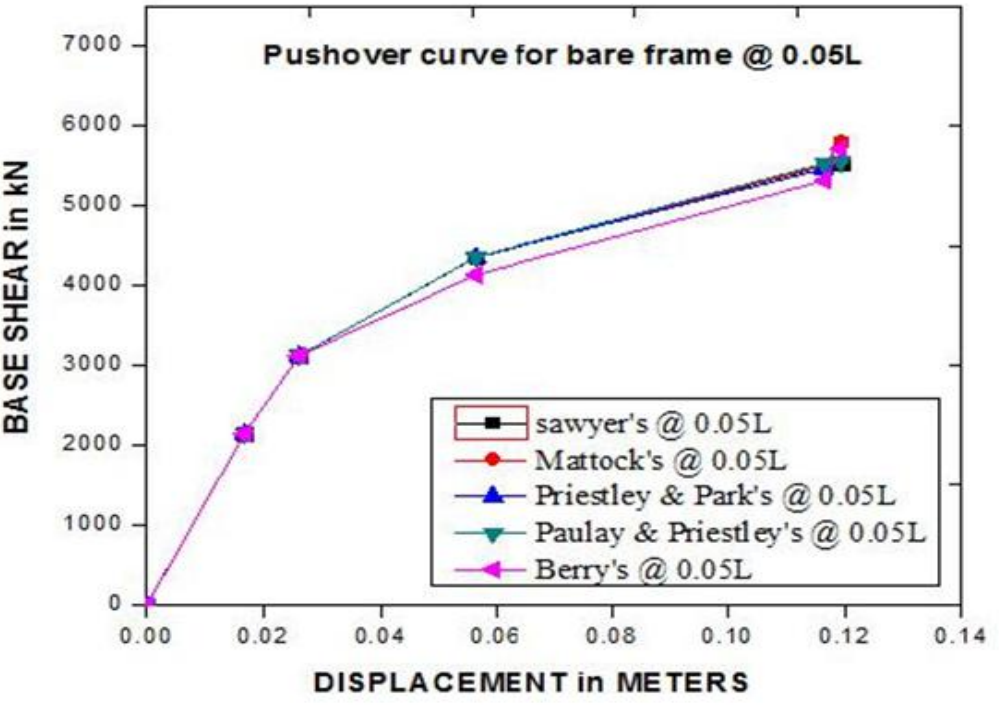


Fig: 5.3.2: Pushover curves for bare frame with hinges at 0.05L

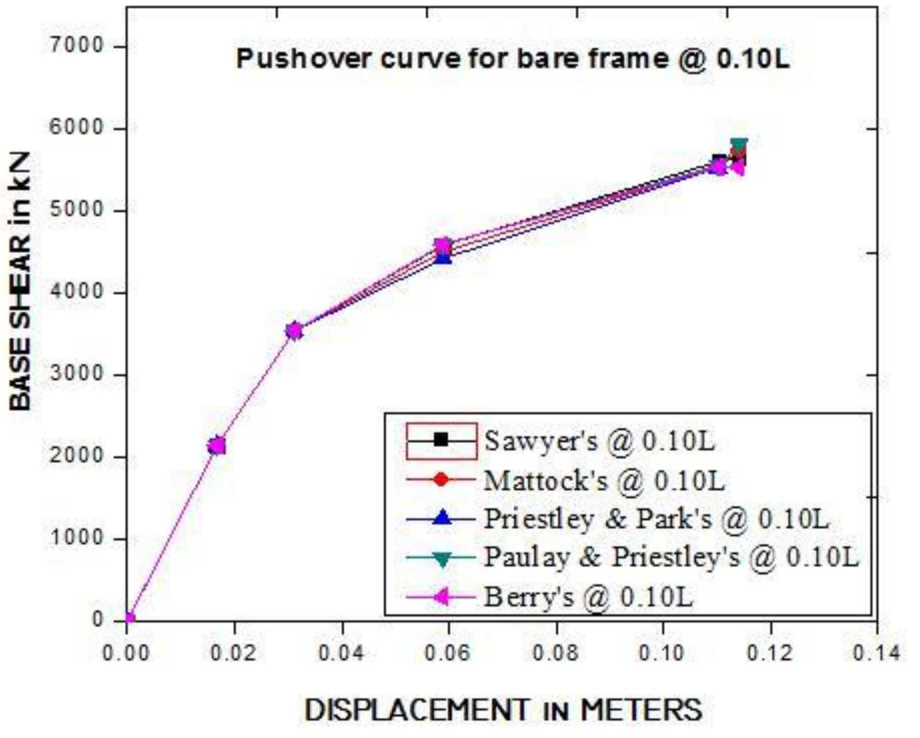


Fig: 5.3.3: Pushover curves for bare frame with hinges at 0.10L

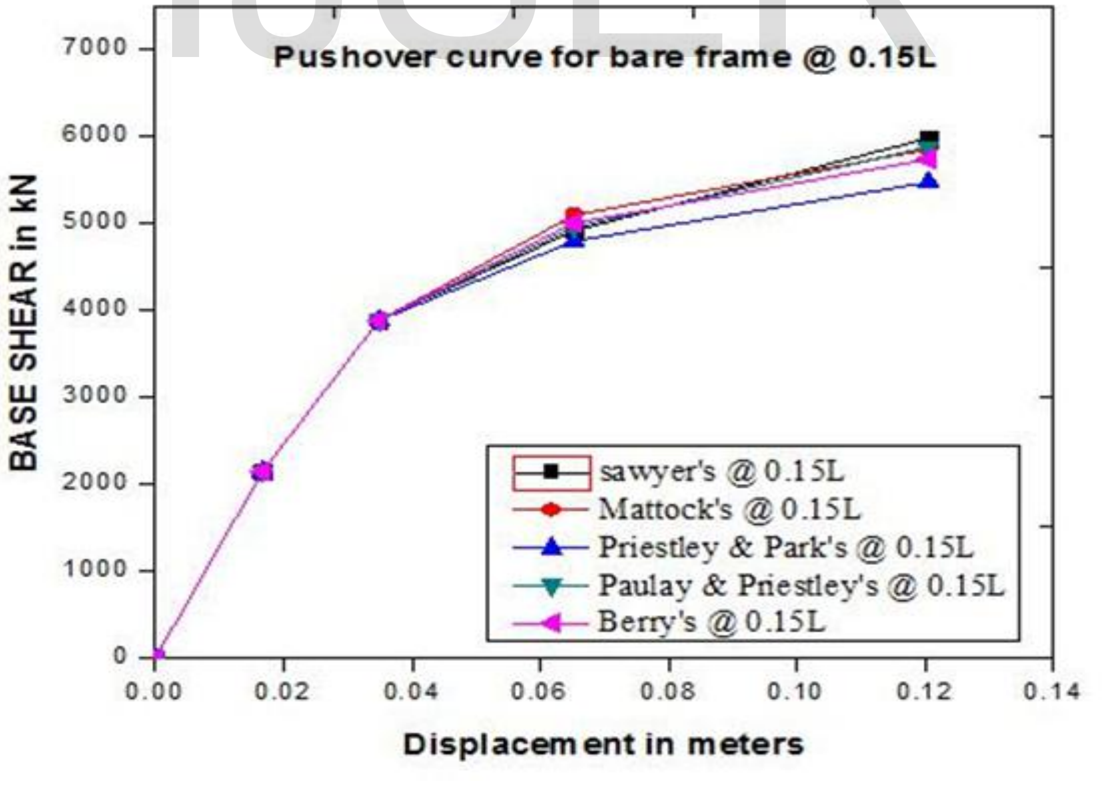


Fig: 5.3.4: Pushover curves for bare frame with hinges at 0.15L

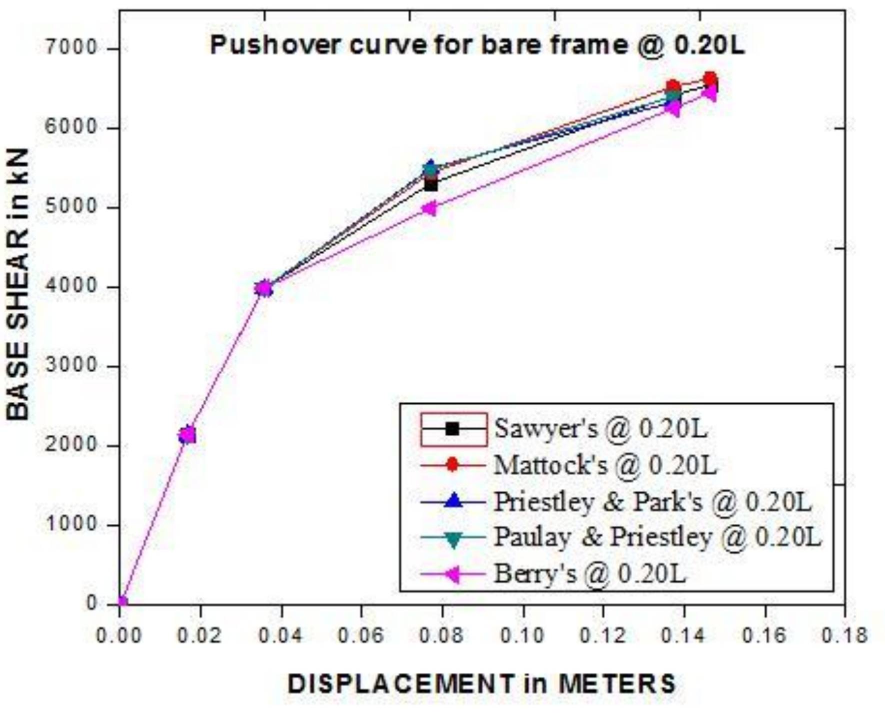


Fig: 5.3.5: Pushover curves for bare frame with hinges at 0.20L

5.4 Comparison of pushover curves for infilled frame with hinges of different hinge length formulations

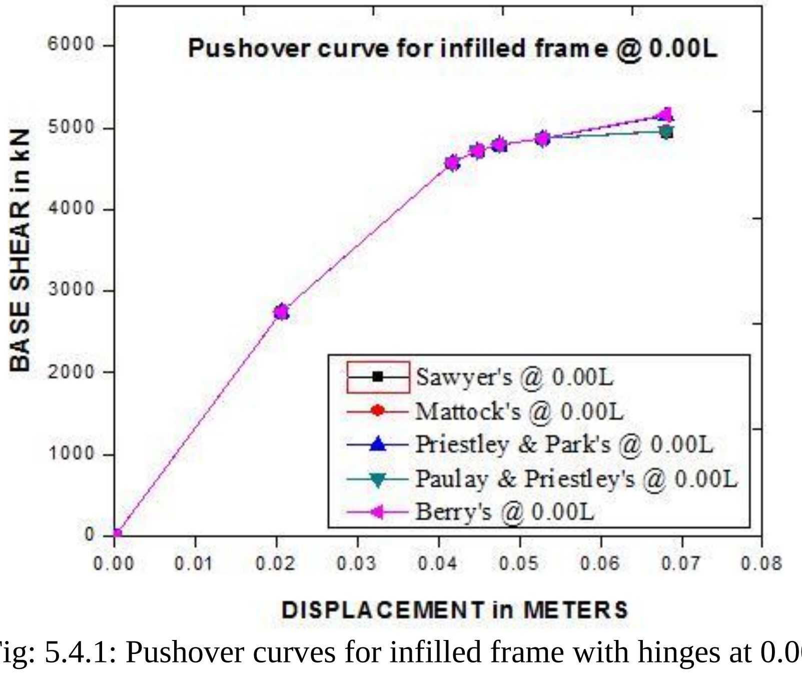


Fig: 5.4.1: Pushover curves for infilled frame with hinges at 0.00L

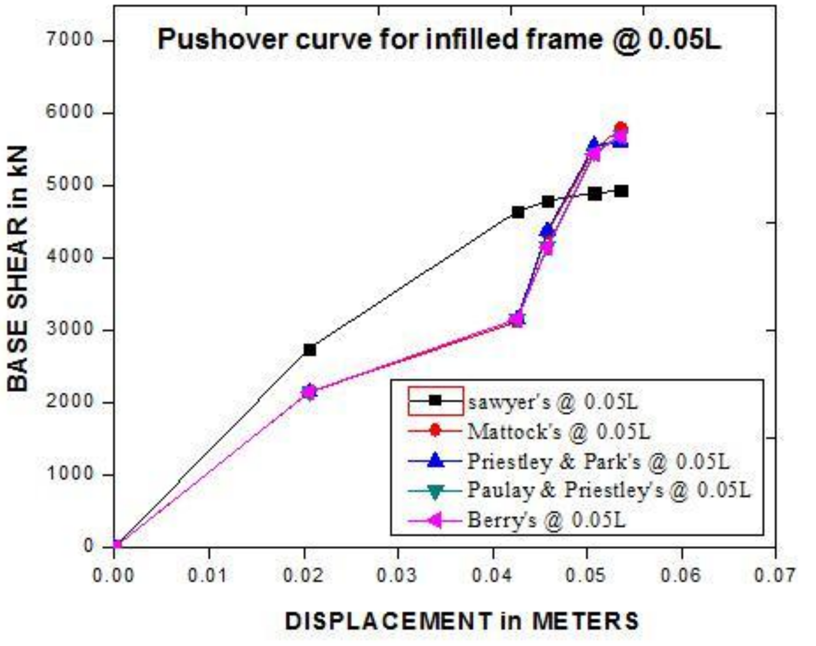


Fig: 5.4.2: Pushover curves for infilled frame with hinges at 0.05L

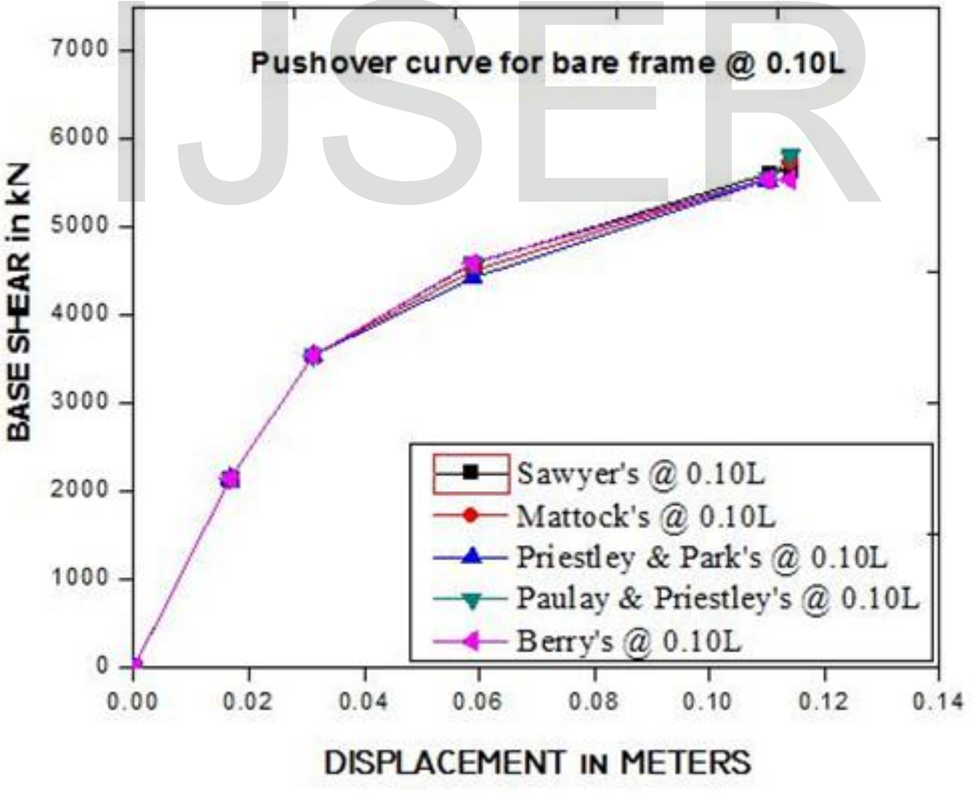


Fig: 5.4.3: Pushover curves for infilled frame with hinges at 0.10L

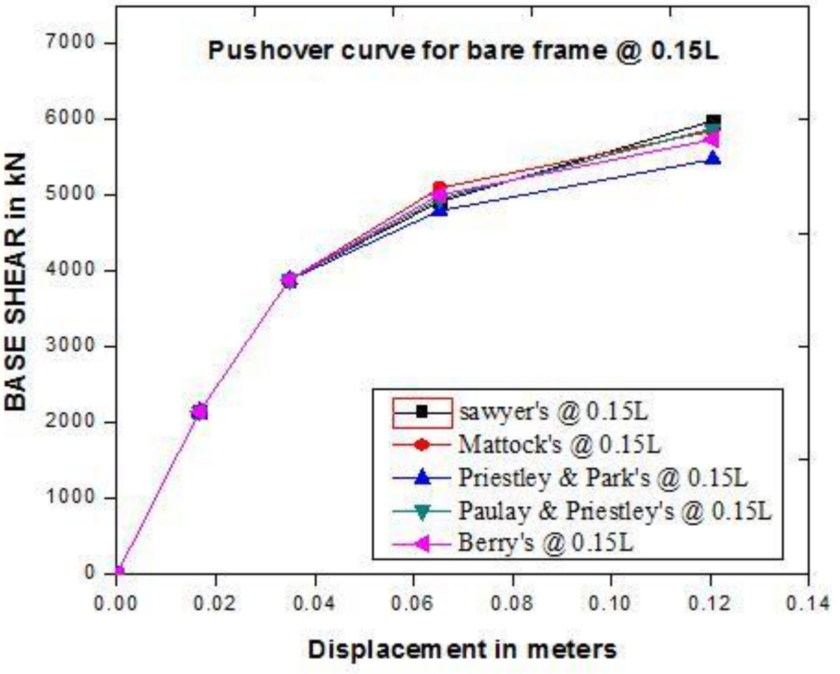


Fig: 5.4.4: Pushover curves for infilled frame with hinges at 0.15L

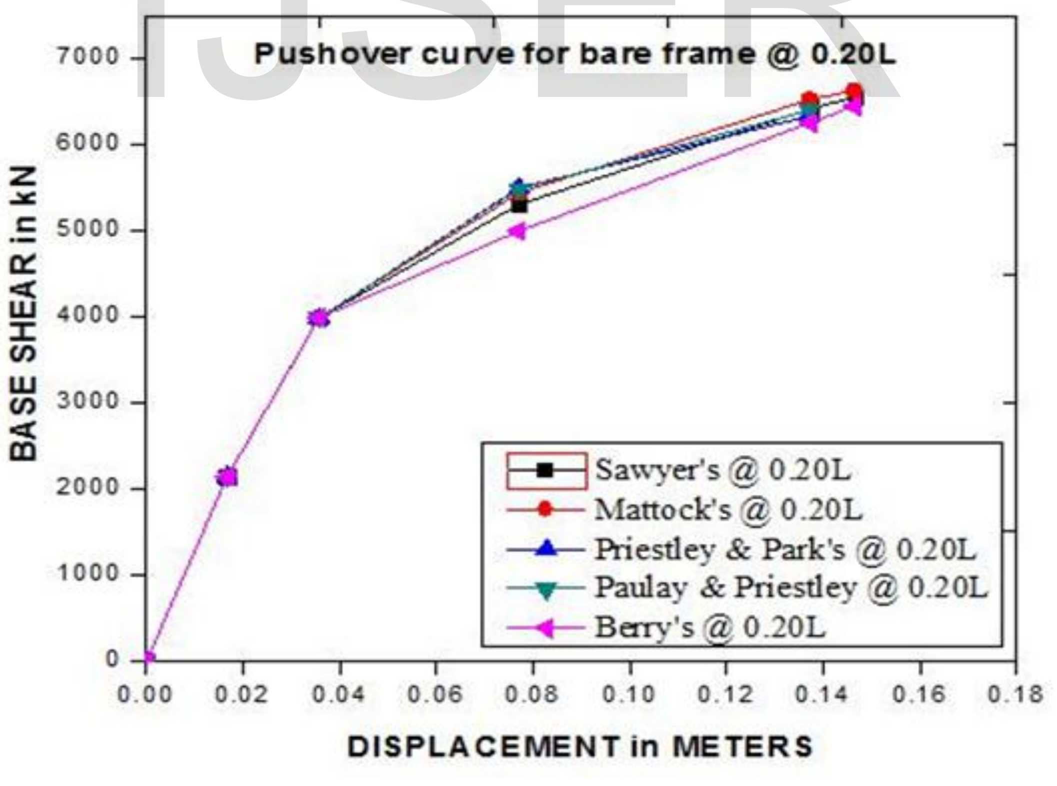


Fig: 5.4.5: Pushover curves for infilled frame with hinges at 0.20L

CHAPTER 6

DISCUSSIONS AND CONCLUSIONS

6.1 Considering the hinge location at support level i.e. at 0.0L as reference for different hinge length formulations.

6.1.1 Sawyer's formula

Table: 6.1.1: Comparison of base shear and displacement values from Sawyer's formula

Location of hinge	Bare frame	Infilled frame
0.0L	P =5295.106kN Δ = 0.109 m	P =5182.725 kN Δ =0.132m
0.05L	0.78% increase in base shear. 0.1% decrease in displacement.	1.46% increase in base shear. 1.51% decrease in displacement.
0.10L	4.13% increase in base shear. 3.66% decrease in displacement.	3.45% increase in base shear. 3.78% decrease in displacement.
0.15L	6.80% increase in base shear. 4.60% decrease in displacement.	6.08% increase in base shear. 3.03% decrease in displacement.
0.20L	9.12% increase in base shear. 5.5% decrease in displacement.	6.20% increase in base shear. 3.03% decrease in displacement.

6.1.2 Mattock's formula

Table: 6.1.2: Comparison of base shear & displacement values from Mattock's formula

Location of hinge	Bare frame	Infilled frame
0.0L	P =5295.106 kN Δ = 0.109 m	P =5188.060 kN Δ =0.132 m
0.05L	0.5% Increase in base shear. 0.91% decrease in displacement.	1.47% increase in base shear. 0.75% decrease in displacement.
0.10L	4.31% Increase in base shear. 3.70% decrease in displacement.	3.44% increase in base shear. 3.78% increase in displacement.
0.15L	7.45% Increase in base shear. 4.58% decrease in displacement.	4.31% decrease in base shear. 3.03% increase in displacement.
0.20L	9.86% Increase in base shear. 6.42% decrease in displacement.	6.31% increase in base shear. 3.03% increase in displacement.

6.1.3 Priestley & Park's formula

Table: 6.1.3: Comparison of base shear & displacement values from Priestley & Park's formula

Location of hinge	**Bare frame**	**Infilled frame**
0.0L	P =3054.968 kN Δ = 0.025 m	P =5237.392 kN Δ =0.131 m
0.05L	0.12% Increase in base shear. No change in displacement.	4.76% increase in base shear. 14.50% decrease in displacement.
0.10L	0.17% Increase in base shear. 4% increase in displacement.	6.37% increase in base shear. 19% decrease in displacement.
0.15L	0.75% Increase in base shear. 4% increase in displacement.	15% increase in base shear. 11% decrease in displacement.
0.20L	1.11% Increase in base shear. 4% increase in displacement.	18% increase in base shear. 10.68% decrease in displacement.

6.1.4 Paulay-Priestley's formula

Table: 6.1.4: Comparison of base shear & displacement values from Paulay & Priestley's formula

Location of hinge	Bare frame	Infilled frame
0.0L	P =5294.827 kN Δ = 0.109 m	P =5763.537 kN Δ =0.136 m
0.05L	0.40% Increase in base shear. 0.91% decrease in displacement.	0.095% increase in base shear. 8.82% decrease in displacement.
0.10L	5.30% Increase in base shear. 3.66% decrease in displacement.	2.09% increase in base shear. 10.29% decrease in displacement.
0.15L	6.91% Increase in base shear. 4.58% decrease in displacement.	3.63% increase in base shear. 13.23% decrease in displacement.
0.20L	9.88% Increase in base shear. 5.50% decrease in displacement.	11.80% increase in base shear. 5.88% decrease in displacement.

6.1.5 Berry's formula

Table: 6.1.5: Comparison of base shear & displacement values from Berry's formula

Location of hinge	**Bare frame**	**Infilled frame**
0.0L	P =5304.071 kN Δ = 0.109 m	P =5462.273 kN Δ =0.113 m
0.05L	1% increase in base shear. 0.91% decrease in displacement.	0.61% increase in base shear. 0.88% decrease in displacement.
0.10L	3.90% Increase in base shear. 3.66% decrease in displacement.	1.51% increase in base shear. 10.61% decrease in displacement.
0.15L	7.52% Increase in base shear. 4.58% decrease in displacement.	7.06% increase in base shear. 5.30% decrease in displacement.
0.20L	9.21% Increase in base shear. 5.5% de crease in displacement.	8.70% increase in base shear. 6.19% decrease in displacement.

Referring the above tables 6.1.1 to 6.1.5 we can say that

- Base shear increases predominantly as the plastic hinges are located away from the supports for both cases of frame configurations.
- Displacement capacity goes on decreases as the plastic hinges are located away from the supports.

- Maximum base shear capacity and increase in base shear capacity as hinges are located away from supports are given by infilled frames because of the rigidity offered by the walls.

6.2 Comparison of the base shear and displacement values of frames with assigned hinges calculated using different hinge length formulations and located at different locations from the support.

Case 1: Bare frame model

(Base Shear in kN and Displacement in meters)

Table: 6.2.1: Base shear & displacement values for bare frame

Hinge location from support / Formula	0.00L	0.050L	0.100L	0.150L	0.200L
Sawyer's	P =5295.106 Δ = 0.109	P =5337.330 Δ =0.108	P =5513.903 Δ =0.105	P =5655.171 Δ =0.104	P =5778.147 Δ =0.103
Mattock's	P =5295.106 Δ =0.109	P =5321.638 Δ =0.108	P =5523.496 Δ =0.105	P =5689.622 Δ =0.104	P =5817.650 Δ =0.102
Priestley & Park's	P =3054.968 Δ =0.025	P =3058.890 Δ =0.025	P =3020.415 Δ =0.026	P =3040.035 Δ =0.026	P =3058.892 Δ =0.026
Pauley-Priestley's	P =5294.827 Δ =0.109	P =5315.530 Δ =0.108	P =5575.122 Δ =0.105	P =5661.175 Δ =0.104	P =5818.163 Δ =0.103
Berry's	P =5304.071 Δ =0.109	P =5357.553 Δ =0.108	P =5511.384 Δ =0.105	P =5703.080 Δ =0.104	P =5792.974 Δ =0.103

Referring the above table 6.2.1 we can say that

- Base shear values for frame with hinges at same locations but formulated by different formulae vary by ± 5%.
- Base shear values for frame with hinges at same locations but formulated by Priestley & Park's are lower than other formulae by about 40 %.
- The hinge lengths calculated using Priestley & Park's formula are low compared to other formulae resulting into lower rotation capacity, hence the lower base shear and displacement capacity.

Case 2: Infilled frame

(Base Shear in kN and Displacement in meters)

Table: 6.2.2: Base shear & displacement values for infilled frame

Hinge location from support / **Formula**	**0.0L**	**0.05L**	**0.10L**	**0.15L**	**0.20L**
Sawyer's	P =5182.752 Δ = 0.132	P =5258.494 Δ =0.130	P =4961.687 Δ =0.137	P =5397.955 Δ =0.128	P =5004.566 Δ =0.136
Mattock's	P =5188.060 Δ =0.132	P =5264.336 Δ =0.131	P =4966.876 Δ =0.137	P =5411.756 Δ =0.128	P =5015.832 Δ =0.136
Priestly& Park's	P =5237.392 Δ =0.131	P =5487.195 Δ =0.112	P =5571.463 Δ =0.105	P =6033.760 Δ =0.116	P =6213.760 Δ =0.117
Paulay-Priestley's	P =5763.537 Δ =0.136	P =5769.047 Δ =0.124	P =5884.248 Δ =0.122	P =5972.970 Δ =0.118	P =6443.781 Δ =0.128
Berry's	P =5462.273 Δ =0.113	P =5495.949 Δ =0.112	P =5545.105 Δ =0.101	P =5848.243 Δ =0.107	P =5937.513 Δ =0.106

Referring the above table 6.2.2 we can say that

- Base shear are high compared to bare frame results in table 6.2.1 because of the rigidity of wall.
- Base shear values for frame with hinges at same locations but formulated by different formulae vary by ± 5%.
- The hinge lengths calculated using Paulay-Priestley's formula are quite high compared to other formulae resulting into higher rotation capacity, hence the higher base shear and displacement capacity.

6.3 Correlation results for bare and infilled frames.

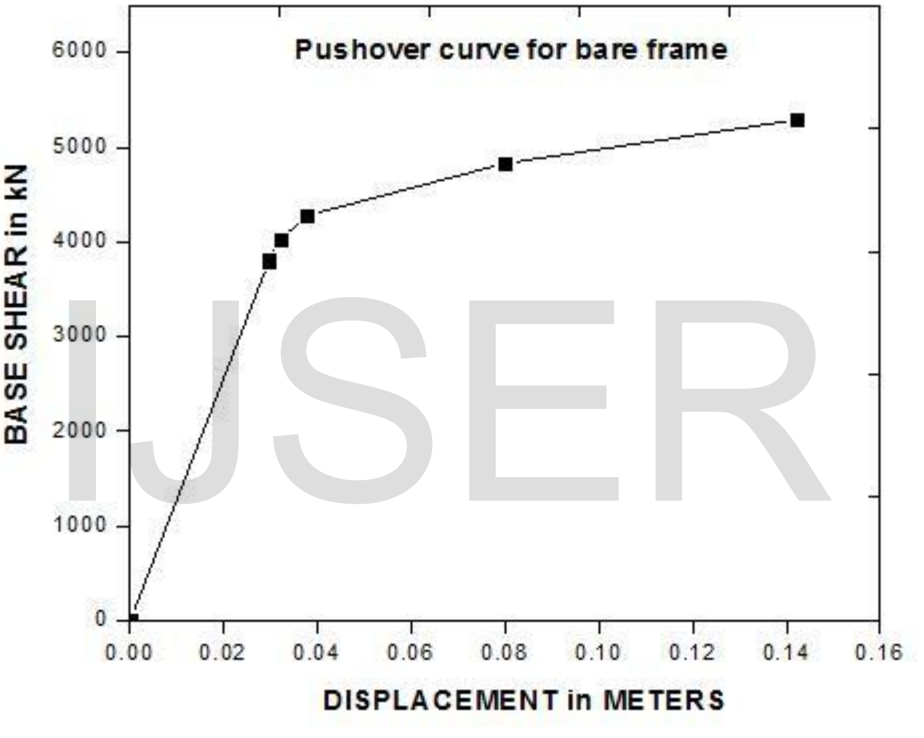


Fig: 6.3.1 Correlation pushover curve for bare frame

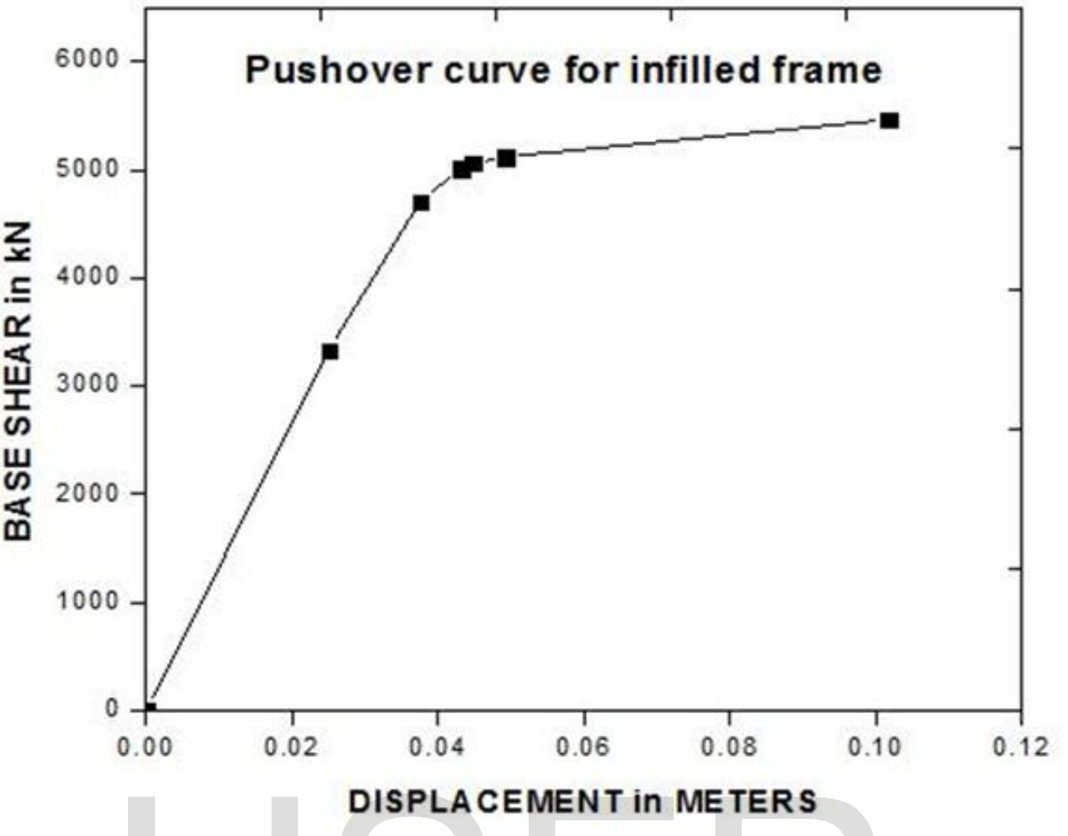


Fig: 6.3.2 Correlation pushover curve for infilled frame

The maximum base shear observed for bare frame is 4986.793KN and the corresponding displacement was found to be 54mm i.e. 0.054m.

The maximum base shear observed for infilled frame is 5535.823KN and the corresponding displacement was found to be 121mm i.e. 0.121m.

Comparing the base shear and corresponding displacement values from correlation results with the analytical results obtained from SAP 2000 analysis for bare and infilled frames as shown in Table 6.2.1 and Table 6.2.2.

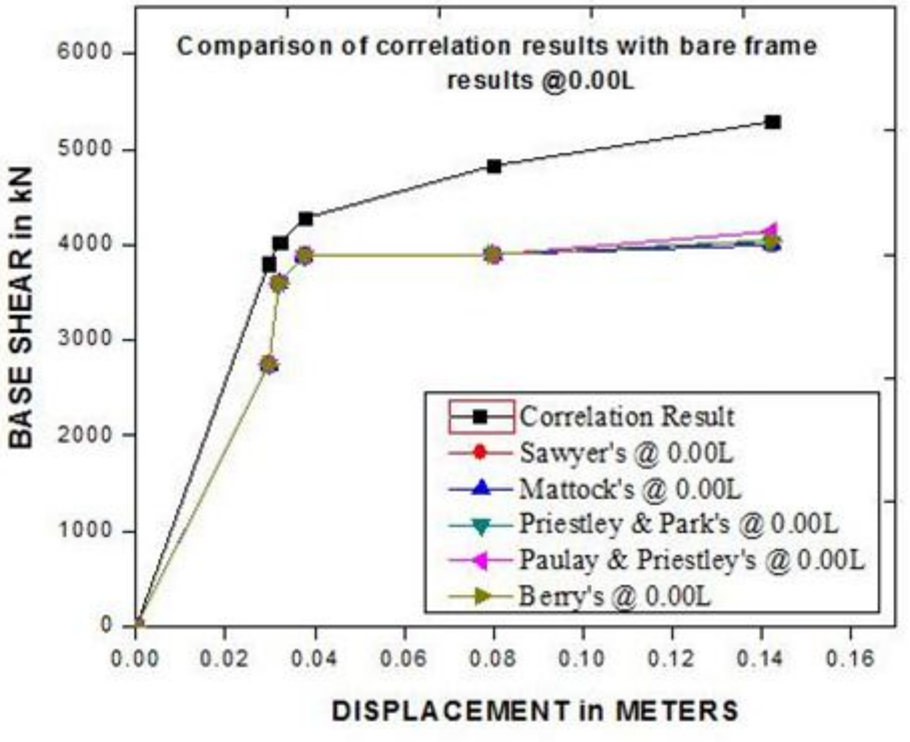


Fig: 6.3.3: Comparison of Correlation results with bare frame results at 0.00L

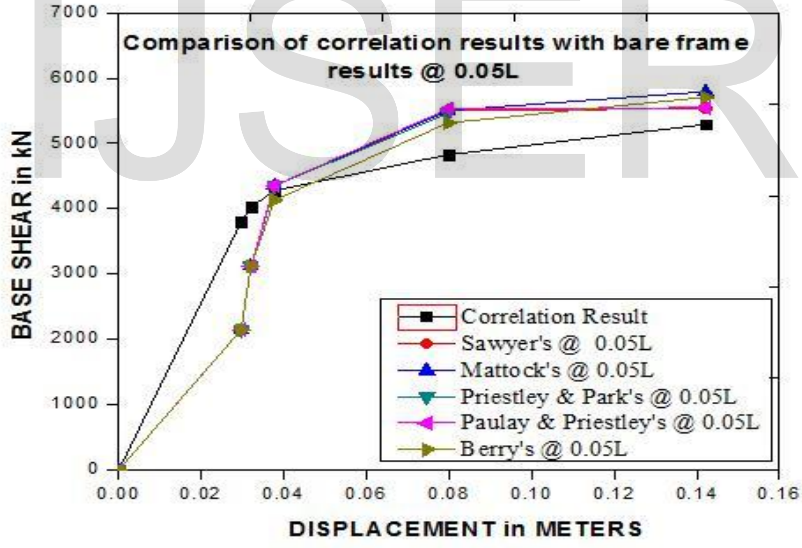


Fig: 6.3.4: Comparison of Correlation results with bare frame results at 0.05L

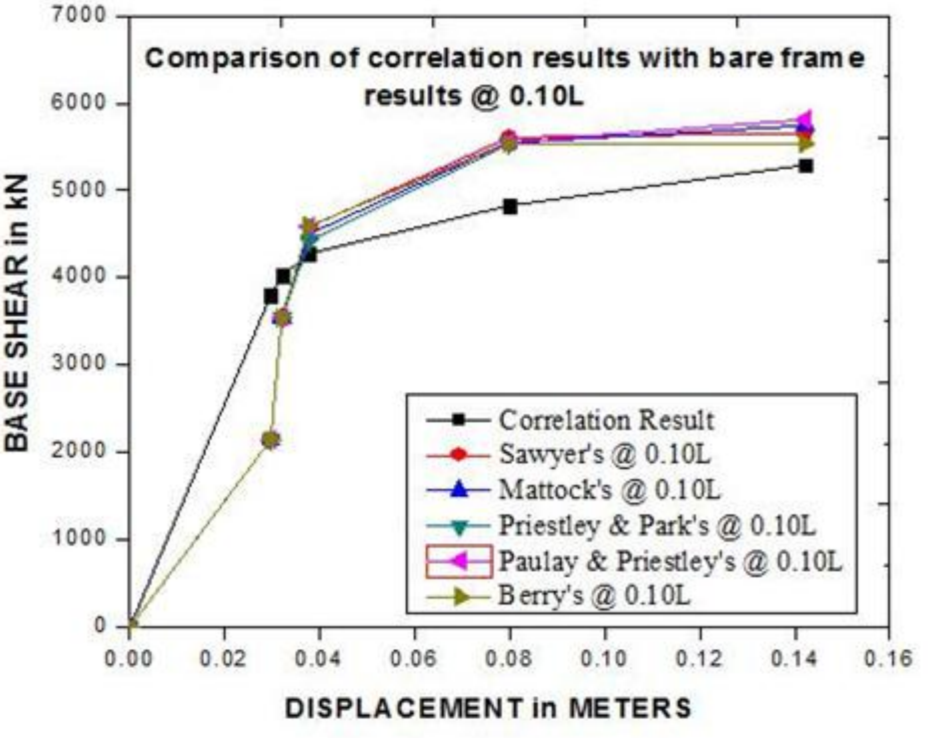


Fig: 6.3.5: Comparison of Correlation results with bare frame results at 0.10L

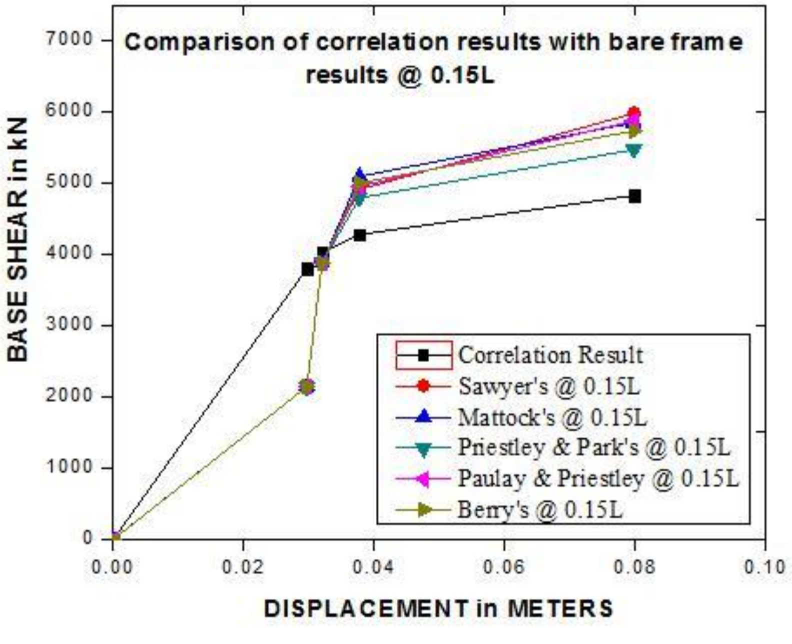


Fig: 6.3.6: Comparison of Correlation results with bare frame results at 0.15L

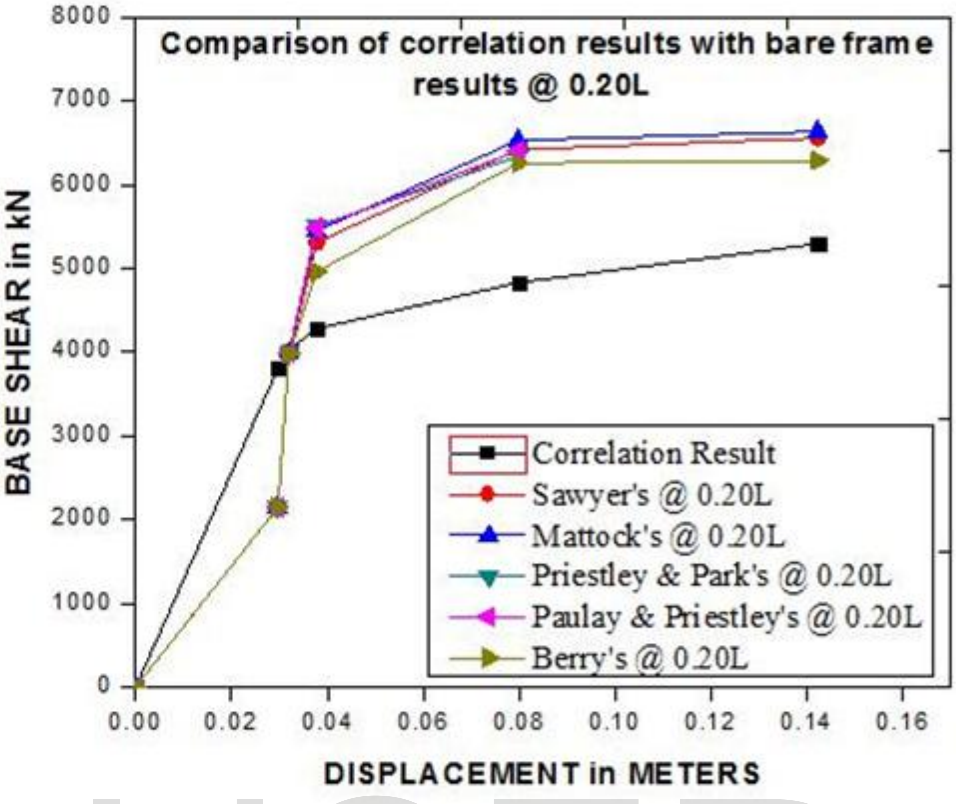


Fig: 6.3.7: Comparison of Correlation results with bare frame results at 0.20L

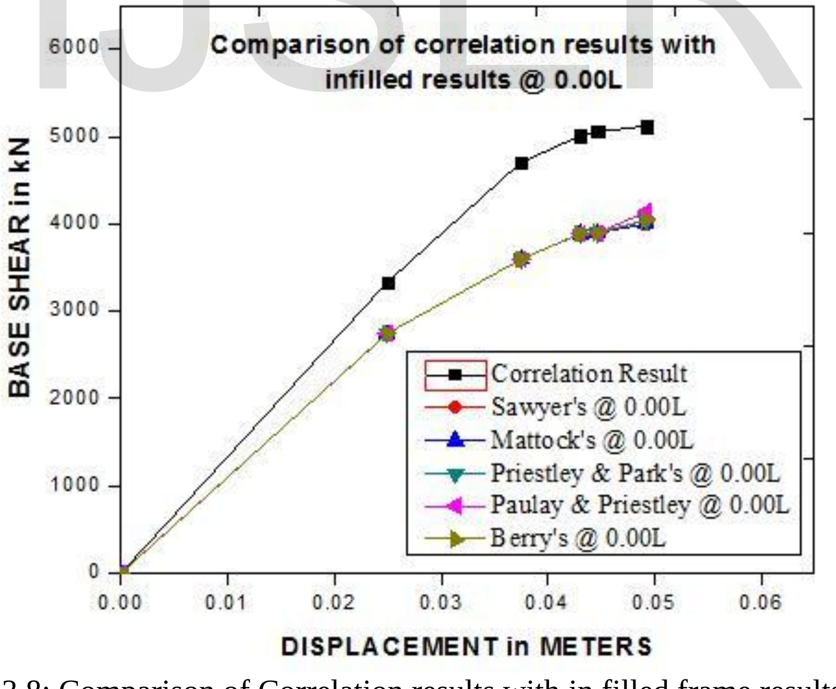


Fig: 6.3.8: Comparison of Correlation results with in filled frame results at 0.00L

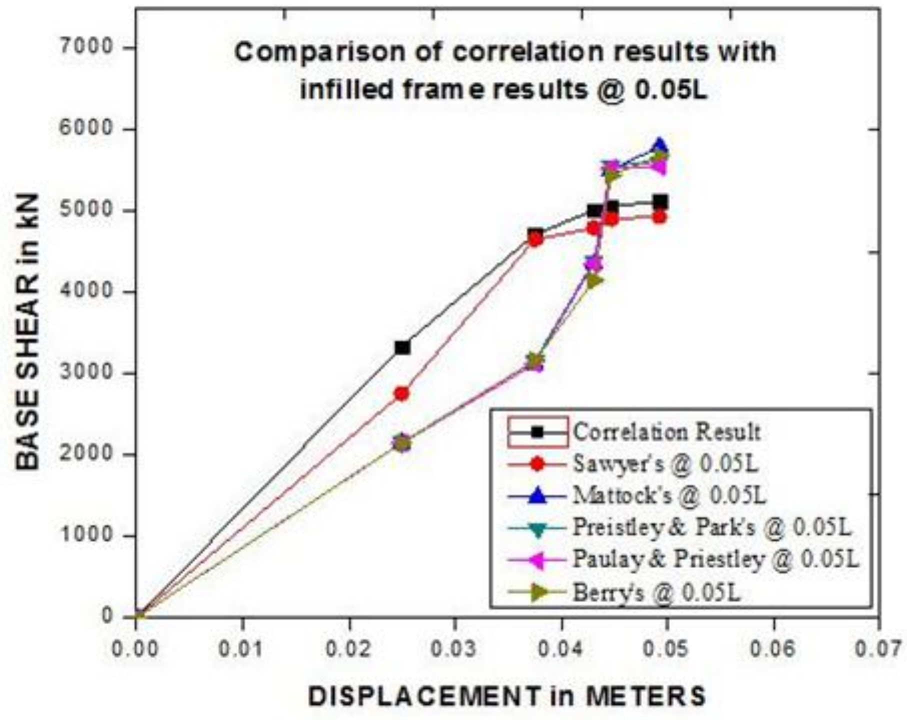


Fig: 6.3.9: Comparison of Correlation results with infilled frame results at 0.05L

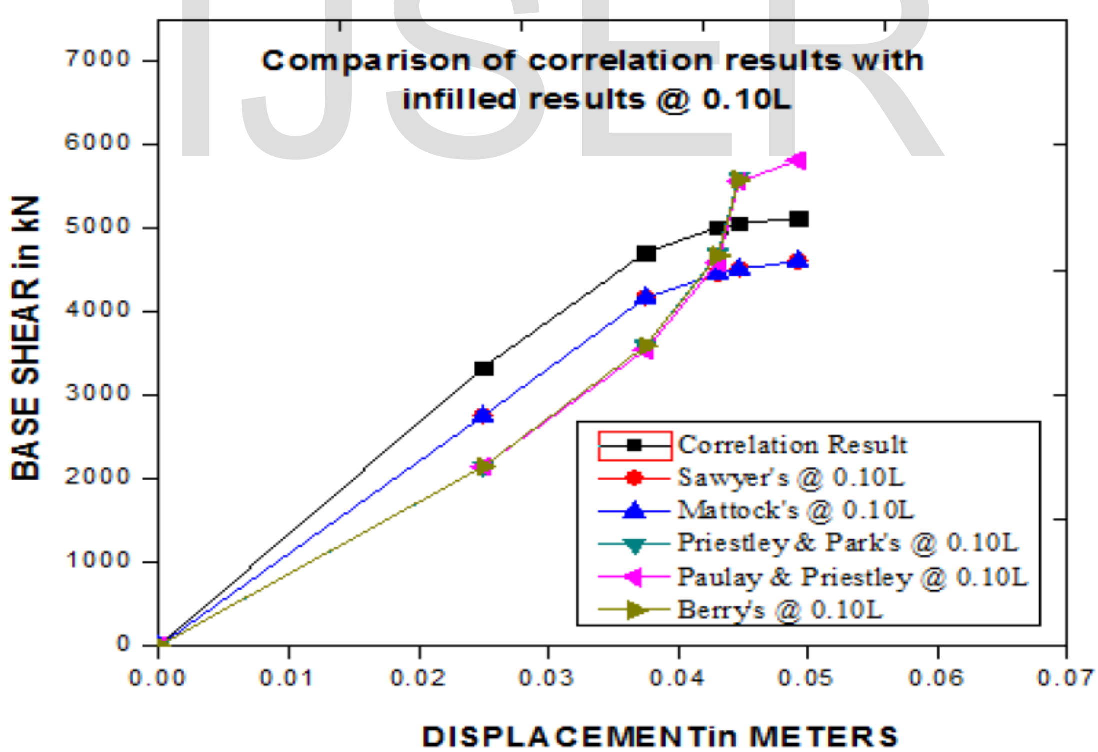


Fig: 6.3.10: Comparison of Correlation results with infilled frame results at 0.10L

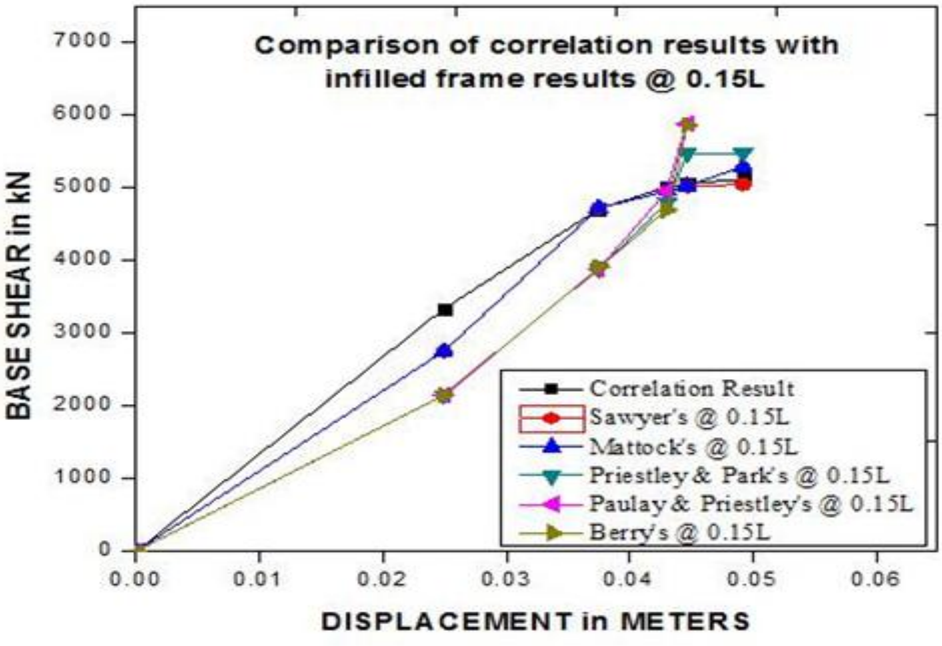


Fig: 6.3.11: Comparison of Correlation results with infilled frame results at 0.15L

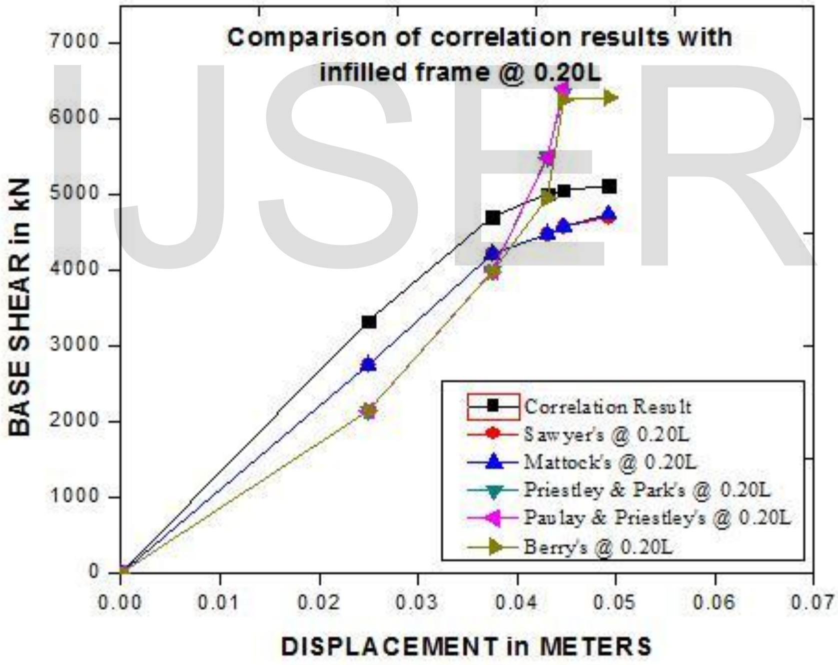


Fig: 6.3.12: Comparison of Correlation results with infilled frame results at 0.20L

Table: 6.3.1: Natural time period values for bare and infilled frames

Mode Number	Natural Time Period	
	Bare frame	Infilled frame
1	1.20322	1.04521
2	1.07387	0.94783
3	1.07387	0.91525
4	0.37746	0.32751
5	0.34108	0.29596

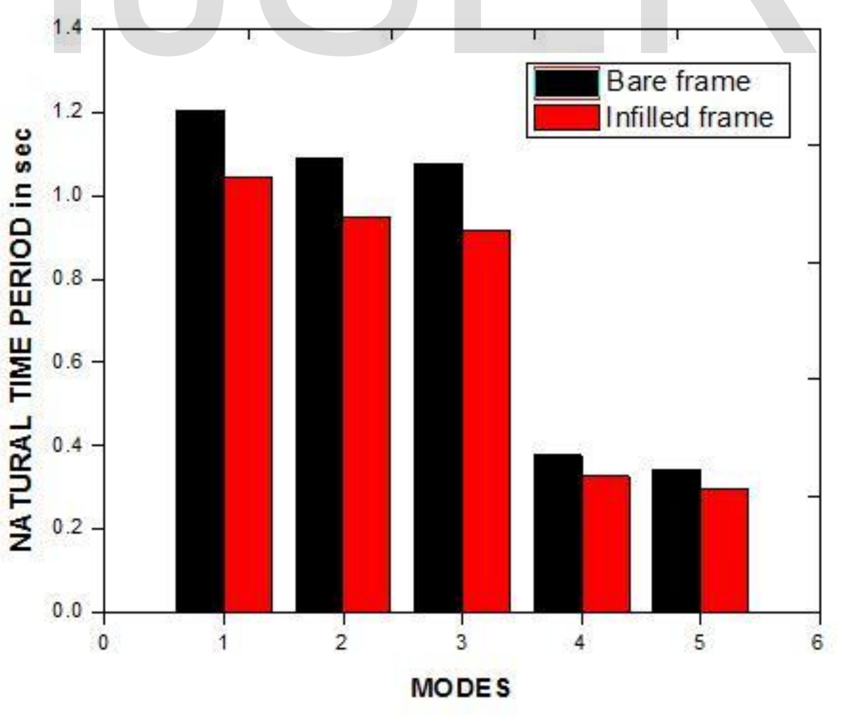


Fig: 6.3.13: Comparison of natural time periods of bare and infilled frame.

CHAPTER 7

CONCLUSIONS

1. Considering infill frame and bare frame, infilled frames have the more stiffness than the bare frame.
2. Base shear increases with the increases in length of the hinge location.
3. The bare frame models with hinges formulated using Priestley & Park's formula shows the 0.61 times lower base shear values than the correlation results.
4. The bare frame models with hinges formulated using Sawyer, Mattock, Paulay and Berry's formula show the higher base shear values than the correlation results.
5. Whereas the corresponding displacement values are high compared to correlation results.
6. The infilled frame models with hinges formulated using Sawyer, Mattock formulae shows the 0.93 times lower base shear values than the correlation values.
7. The infilled frame models with hinges formulated using Priestley; Paulay and Berry's formula shows the 1.03 times higher base shear values than the correlation values.
8. The corresponding displacement values are high compared to the correlation results.
9. Base shear values for analytical results are significantly more than correlated results for both bare and infilled frames.
10. Natural time period for the bare frame is 1.5 times greater than the infilled frame.

6.4.1 SCOPE FOR THE FURUTRE STUDY

Present study is lacking from other consideration, which are neglected and some of the other important parameters are not considered, following are the points which are necessary for further study.

- In current study, plastic hinge length is calculated using Sawyer, mattock, Priestley, paulay and Berry's formula. Plastic hinge length can also be calculated using other formulas for further study.
- The study can be carried out using nonlinear dynamic analysis, (time history analysis) for accurate results.
- The study can be carried out using response spectrum method for further study.
- Variation of response reduction factor can also be taken into account for further study.
- Variation of ground location can also be taken for the further study.

APPENDIX A

Table: A1 Detailed data of building model studied

Storey Height	3m in all floors
Building Type	Commercial
Type of Foundation	Isolated
Seismic Zone	V
MATERIAL PROPERTY	
Concrete Grade	M_{30}
Steel Grade	Fe 415
Elastic modulus of steel	2*10^8 kN/m^2
Elastic modulus of concrete	2*10^5 kN/m^2
Elastic modulus of Brick Masonry	13.2*10^6 kN/m^2
Density of concrete	25 kN/m^2
Density of Masonry	20 kN/m^2
MEMBER PROPERTY	
Thickness of slab	100 mm
Beam	(250*500) mm
Interior Column	(500*500) mm
Exterior Column	(250*500) mm

Thickness of wall	250 mm
ASSUMED DEAD LOAD INTENSITY	
Floor finish, DPC	2 kN/m^2
FBBM	3*0.25*20 = 15 kN/m^2
PPT	0.9*0.25*20 = 4.5 kN/m^2
LIVE LOAD INTENSITY	
Roof	0.75 kN/m^2
Floors	3 kN/m^2
Earthquake live load on slab as per IS: 1893(Part 1)-2002	
Roof	0
Floors	0.25*3 = 0.75 kN/m^2

Equivalent Static Analysis as per IS: 1893(PART 1)-2002

Table A2: Input data to the Building for Equivalent static analysis

Zone	V
Zone Factor	0.36
Importance factor *I*	1
Response reduction factor	5
Damping ratio	5%

Fundamental Natural Period:

Approximate natural period of vibration for moment resisting frame with and without brick infill panels are calculated using empirical equation given below.

$T_a = 0.075*h^{0.75}$ (for RC frame building without brick infill)

$T_a = (0.09*h)/\sqrt{d}$ (for RC frame building with brick infill)

Where, h = height of building in meter

d = base dimension of building at plinth level, in meter along considered direction

Fundamental natural time period in longitudinal and transverse direction

For bare frame model

$T_a = 0.075*15^{0.75} = 0.6$ S

For infill frame model in longitudinal direction

$T_a = (0.09*15)/\sqrt{18} = 0.32$ S

For infill frame model in transverse direction

$T_a = (0.09*15)/\sqrt{30} = 0.246$ S

Response acceleration coefficient,

For medium soil,

$$(S_a/g) = \begin{cases} 1+15T & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.55 \\ 1.00/T & 0.55 \le T \le 4.0 \end{cases}$$

Design Horizontal seismic coefficient, $A_h = (Z/2)\ (I/R)\ (S_a/g)$

$= 0.09$

Design seismic base shear,

$V_B = A_h$ x W

V_B = 0.09*65070

= 1626.75 KN

For Z= III, V_B = 0.04*65070

= 2602.8 KN

Vertical distribution of base shear along the building height.

The Design base shear V_B computed is distributed along height of the building as pe

$$Q_i = V_B \left[(W_i h_i^2) / \sum_{j=1}^{n} Wi\ hi^2 \right]$$

Table: A3 Input data for strut width calculation for exterior column

STRUT WIDTH CALCULATION USING EMPIRICAL EQUATION	
Beam	(0.25*0.5) m
Column	(0.25*0.5) m
I_c	$(0.25*0.5^3)/12 = 0.002604\ m^4$
I_b	$(0.25*0.5^3)/12 = 0.002604\ m^4$
Elastic modulus of Masonry (Em)	13800 N/mm^2
Thickness of wall	0.25 m
Centre to Centre of wall (H)	3-0.25-0.25 = 2.5 m
Centre to Centre of column (L)	6-0.25-0.25 = 5.5 m