SEISMIC ANALYSIS OF RCC BUILDINGS WITH AND WITHOUT MASONRY INFILLS

A

DISSERTATION

Submitted in partial fulfillment of the

requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

EARTHQUAKE ENGINEERING

(With Specialization in Structural Dynamics)

By

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CANDIDATE'S DECLARATION

I hereby declare that the work which is being presented in this dissertation report entitled **"Seismic Analysis of RCC Buildings with and without Masonry Infills",** in partial fulfillment of the requirements for the award of the Degree of **Master of Technology** in **Earthquake Engineering** with specialization in **Structural Dynamics**, submitted to the Department of Earthquake Engineering, Indian Institute of Technology Roorkee, is an authentic record of my own work carried during the period from July 2015 to May 2016 under the supervision of **Dr. R. N. Dubey**, Assistant Professor, Department of Earthquake Engineering, IIT Roorkee, Roorkee.

The matter embodied in this report has not been submitted by me for the award of any other degree or diploma.

Place: Roorkee (**Someraj Ray)**

Date:

CERTIFICATE

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

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(Someraj Ray)

ABSTRACT

The presence of infill in reinforced concrete structure can extremely affect the response of structure under seismic loads. There might be a positive effect due to increase of strength and stiffness but the drawbacks start to appear due to stress concentrations.

The codes have limited information on the design of infilled structures besides different architectural requirements which increases the problems. Pushover analysis was performed on the structure to study the influence of infill on the performance of RCC structures.

The results show that the influence of infill on the structural performance is significant. The structural response such as fundamental period, roof displacement, inter story drift ratio, stresses in structural member reduces with the incorporation of infill.

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CHAPTER-1

INTRODUCTION

1.1 GENERAL

The potential for interaction of infill walls and partitions with the structural system has often been ignored to simplify the design or because the lack of design information has made it difficult to assess the extent of composite action. There are two very important reasons why this practice is not satisfactory [1]. Firstly, in today's competitive market, the choice of the structural system for a building may be largely determined by the efficiency of transmitting lateral loads to the foundations, particularly for multi-storey buildings in high wind or seismic prone areas. In most cases, the efficiency is related to the ability of the structural system to limit the inter-storey drift of the building under lateral loads. Ignoring the substantial stiffening effect of infill walls can lead to an inefficient and uneconomical design of the structural frames where both the strength and stiffness requirements for the frame could be substantially reduced.

Secondly and perhaps the most important issue is, neglecting the influence of infill walls would not always be a move towards conservative design. Infill wall can make a flexible frame very stiff. It can also ominously disturb the distribution pattern of the lateral load to different components of a structure. Thus, higher loads, than expected, may be attracted to an infilled section, possibly leading to cracking of the wall. In addition the frame will be overstressed .Therefore, the interaction of the infill wall in association with the frame should be considered in order to achieve an efficient design and to ensure that neither the wall nor the frame is overstressed. Similarly the interaction of the partition wall with the surrounding frame may lead to an inefficient design of the frame and /or cracking of the partition wall [1].

In this study, a building has been selected and designed as an SMRF building based on Indian codal provisions. Various strut models representing infill walls have been considered and compared. Initially, in plane action of bare and infill frame has been considered by response spectrum method of analysis, after that pushover analysis has been performed and performance point of the buildings have been calculated.

1.2 TYPES OF DESIGN APPROACH

1.2.1 Force-based design

Traditionally, codes all over the world, were based upon force based design concept. In this concept, every sole participant of the building is designed for strength so that it can bear the load it would be subjected to. Inelastic effects are taken into account by assuming a flat value of response reduction factor. Buildings are designed so that they can endure shaking during moderate intensity tremor and to avert total collapse during high intensity earthquake. Flexibility is controlled by putting restrictions on maximum inter-storey drift. Minimum base shear is controlled by the use of capping on design natural time period of the buildings.

But this method has many disadvantages too. Many researchers have specified out that force is a very poor index of damage and there is no explicit relationship between strength and damage [12]. Also presuming a flat value of response reduction factor is not optimistic because ductility depends upon many parameters such as percentage of steel, axial force, degree of redundancy, structural geometry etc.

1.2.2 Displacement based design

Displacement Based Design was first presented by Qi and Moehle (1991), to eliminate the shortcomings of the force based design. Its elementary design criteria included strain, rotation, translational displacement etc. In this process, design principles are expressed in terms of attaining a set of performance objectives. Priestley (2000) has made substantial impact in evolving a rational technique for displacement based design. In this methodology, control parameters were inter-storey drift and ductility demand for attainment of a required performance.

A performance objective characterizes a specific level of risk. This methodology offers the building owners' and policy makers' a framework for well-versed judgement about tolerability of seismic risk.

A genuine assessment of strength and ductility is done and performance levels are regulated in terms of inelastic deformations in different components. This is done so because inelastic deformations are the finest index of damage. It takes into account performance criteria by appropriate assessment of demand and capacity by pushover analysis.

1.2.3 IS code design philosophy

The Indian code has two levels of seismic hazards, the first one being Maximum considered earthquake (MCE) and the second one being Design basis earthquake (DBE)[6]. Structures are generally designed for DBE. We can determine the design horizontal seismic coefficient Ah by using the expression given below

$$A_h = \frac{ZISa}{2Rg} \quad \text{.........(1.1)}$$

where;

Z = Zone factor meant for the Maximum Considered Earthquake (MCE)

I = Importance factor. The usefulness of the structure determines the importance factor.

R = Response reduction factor

Sa/g = Average response acceleration coefficient corresponding to the natural time period of the structure.

1/2 = The factor used to obtain Design Basis Earthquake (DBE) from MCE

The total base shear acting upon the structure is given by the subsequent equation given below;

$$V_b = A_h * W \quad \text{.........(1.2)}$$

where, W is the total seismic weight of the building.

1.3 METHODS OF ANALYSIS

1.3.1 Equivalent static method

In this method, the total base shear which is acting upon the structure is distributed accordingly throughout the height of the building. The seismic coefficient is used to calculate the base shear acting upon the building. The seismic coefficient depends upon two factors, first one being seismic exposure of a given location and second one being the total mass of the structure. Though this approach is a static approach, the dynamic properties are also taken care of in terms of incorporation of fundamental time period and response reduction factor. For structure for which the first mode of vibration commands the maximum response, only for those structures this method can be used. If during modeling and analysis, effect of infill is taken into account, then most of the structure would give their maximal response at the fundamental mode of vibration. This happens due to the fact that infills make the structure more rigid and stiff.

1.3.2 Response spectrum analysis method

In this method, the peak modal responses that are obtained from the dynamic analysis of a single degree of freedom system are used. Response spectrum curve which is a plot of spectral acceleration v/s period is plotted by finding out the peak acceleration at different periods for the model. The curve which is obtained is very rough, but code prescribes to use a smoothened curve. For low range of periods, the values are kept constant but for high period, it is varied. If we possess the site specific spectrum, then we don't need to use the code specified spectrum.

For multi-degree of freedom systems, this method is extended by carrying out linear superimposition of mode shapes by means of the modal combination methods for instance SRSS (square roots of the sum of squares) and CQC (complete quadratic combination). The major disadvantage of SRSS combination is that it does not consider the effect of closely spaced modes as is taken care of in CQC method of combination. For desired damping values, the results of this analysis provide the user with only the peak structural response.

1.3.3 Pushover analysis method

This is an approximate method of analysis in which the structure is subjected to steadily increasing lateral forces with an invariant height-wise distribution. This process continues till a target displacement is achieved. This method consists of a succession of sequential elastic analysis which is superimposed to obtain approximately a force-displacement curve of the entire structure. Firstly, a two or three dimensional model is created which has all bilinear or trilinear load deformation figures of all the lateral load resisting elements. After that, the gravity loads are applied. A predefined lateral load arrangement which is distributed along the building height is then applied. Until some members start to yield, the lateral force are continued to increase. As the stiffness of yielded member reduces, the structural model is altered to account for it and the lateral forces are once again increased, until some more members again get yielded. The entire process is continued with unless a control displacement at the top of structure reaches a definite level of deformation or structure becomes globally unstable. The global capacity curve is obtained by plotting a curve between the roof displacement v/s the base shear.

Pushover analysis can be performed in two ways. First one is force-controlled and second one is displacement controlled. Full load combination is applied upon the structure in force-controlled mode. This method should be applied only when the load such as gravity loading is known. Where there is not much knowledge about the intensity of applied load in advance, we continue to increase the magnitude of applied loads until we get the target displacement or the structure develops a collapse mechanism. Commonly, roof displacement is selected as the control displacement.

After performing the above analysis, the internal forces and the strains that we get are used as approximations of inelastic strength. After that the available capacities are compared with deformation demand for performance check.

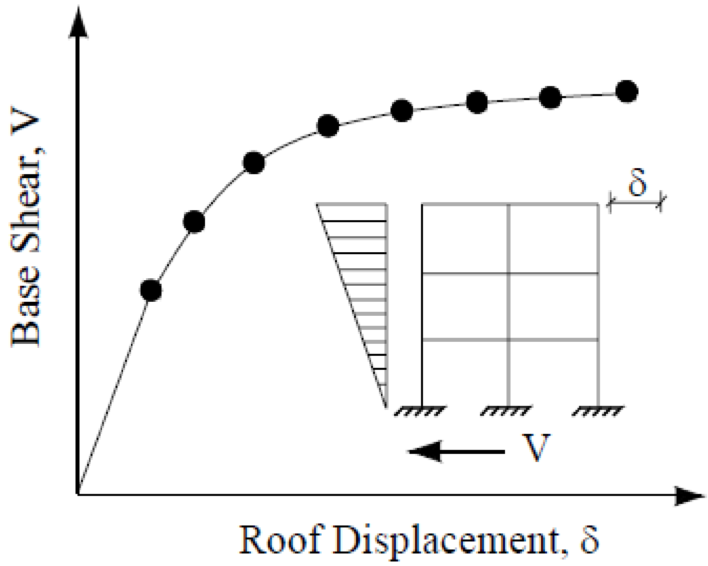


FIG. 1.1. A generalized Pushover curve of a building [10]

1.3.4 Time history analysis

It is the most accurate, best and reliable method of analysis for a structure. In this method, a time history is applied on the structure i.e. a real ground motion record; and feedback of the structure is calculated at every incremental time step. With each increasing time step the response of the structure in terms of displacement and or stresses is calculated. Every earthquake has its unique time history which changes with every recording station due to change in amplitude and frequency. The availability of good records is a drawback.

1.4 OBJECTIVE AND SCOPE OF PRESENT STUDY

1. To compare different methods which are used for modelling the infills and calculating its width of equivalent diagonal strut.
2. To compare the time period of the building with different strut models and IS code method.
3. To perform pushover analysis of the representative buildings.
4. To perform IDA for the selected building.
5. To plot the fragility curves for the building chosen.

CHAPTER-2

BEHAVIOUR OF URM UNDER IN-PLANE ACTION

2.1 IN PLANE ACTION

When the lateral forces are acting upon the structure, the action of infill is like a diagonal strut attached to the frame on its compression side and disconnected from the frame on its tension side. Owing to the high stiffness and diagonal strut action of the infill it bears a part of the lateral load acting on the structure.

2.2 FAILURE MODES

Due to its high stiffness and strength, the infill carries part of the lateral load acting upon the structure due to which cracks start to appear in the infill and the infill gets separated from the frame only leaving two of its corners under compression in contact with the frame. This leads to the diagonal strut action of the infill. Due to this strut action, various failure modes are generated depending upon the strength of masonry and frame material [1].

(a) Corner crushing mode (CC mode) – In this mode, due to high localized stresses prevailing at one of the loaded corners, the corners of the infill gets crushed. It is generally associated with weak masonry infill encompassed with infirm joints and firm members.

(b) Sliding shear failure mode (SS Mode) - Bed shear sliding along joints have been identified as frequent absolute failure modes observed in infills during earthquakes. It shows the horizontal drifting shear failure of the bed joints of a masonry infill. This mode of failure is generally associated with strong frame and feeble mortar joints.

(c) Diagonal compression mode (DC mode)- In this mode the infill in its central region gets crushed. It happens in infilled frame with a slender infill, due to out of plane buckling failure of the infill.

(d) Diagonal cracking mode (DK mode) - This failure mode is associated with the formation of a crack joining the two loaded corners. It occurs when a strong infill is present in a weak frame.

(e) Frame failure mode (FF mode) - In this mode of failure, plastic hinges are formed in the column due to low shear carrying capacity of the columns. It is generally associated with frame with weak joints with strong infill or with weak frame.

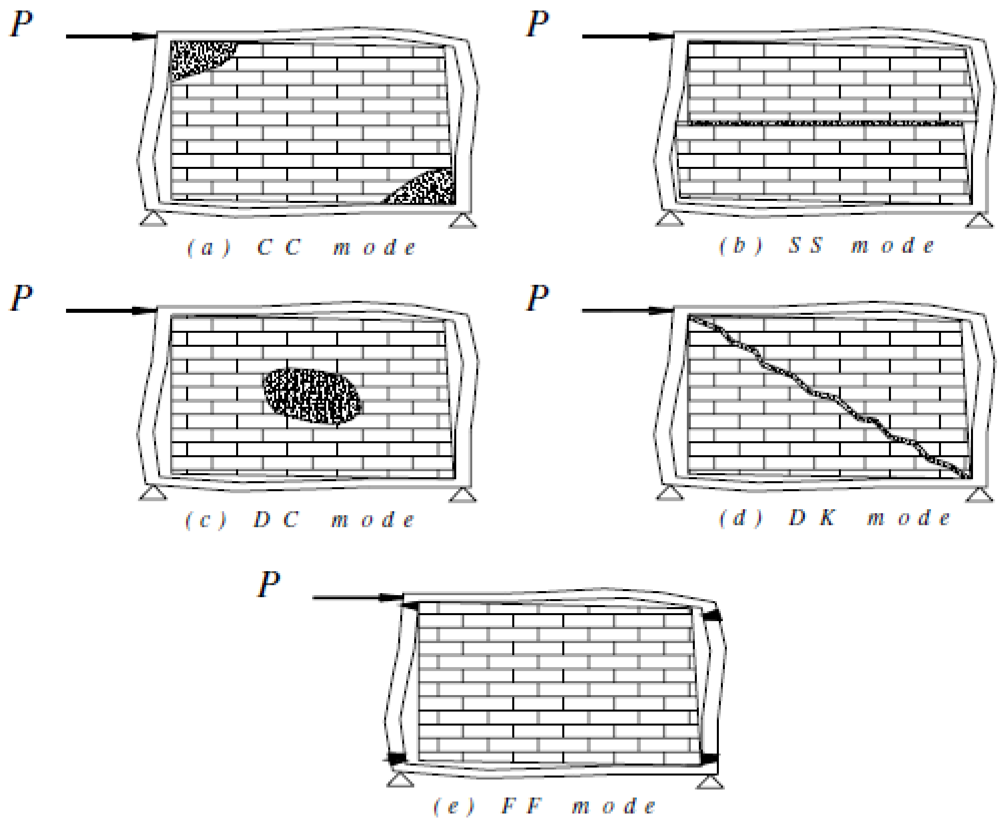


FIG.2.1 Different failure modes of in plane masonry under in plane action [3]

CHAPTER-3

FRAGILITY FUNCTION AND GROUND MOTION SELECTION

3.1 FRAGILITY FUNCTIONS

Fragility curve of damage sustained by the building of a given class is a plot of the probability of being exceeded at a given intensity measurement. The probability of being in, or exceeding, a given damage state is modeled as a cumulative probality distribution function. It is given by

$$P[d_s/S_d]=\Phi[\frac{1}{\beta}\ln(\frac{S_d}{S_{d,ds}})] \quad \text{......(3.1)}$$

where,

$S_{d,ds}$= Median interstorey drift for damage state d_s

Φ = Normal cumulative disribution function

β =Total uncertainity

$$\beta = \sqrt{\beta_m^2 + \beta_D^2 + \beta_C^2} \quad \text{......(3.2)}$$

where

β_m is the modelling dispersion uncertainity. It involves uncertainities which are associated with the capacity of building. It takes into account quality and completeness of mathematical modelling and it varies with type of building.

β_D is the demand uncertainity. Each ground motion is unique depending on frequency content, duration, seismic source, path attenuation and the site effect of the ground motion. Even for same PGA of ground motion the response of same building under different earthquake will be different.

β_C is the capacity uncertainity. It is related to capacity of the building.

3.2 SELECTION OF GROUND MOTION

The earthquake ground motion has been selected according to the criteria laid down by FEMA P695 which suggests selection of earthquakes based upon source distance, magnitude, type of fault rupture and instrument capablity.

TABLE 3.1 Ground motion suite

S.No.	Event	Year	Station	Magnitude	Mechanism	Epicentral Distance(km)
1	Northridge	1994	Canyon Country-WLC	6.7	Thrust	26.4
2	Duzce, Turkey	1999	Bolu	7.1	Strike Slip	41.3
3	Hector Mine	1999	Hector	7.1	Strike Slip	26.5
4	Imperial Valley	1979	El Centro Array #11	6.5	Strike Slip	33.7
5	Kobe, Japan	1995	Nishi-Akashi	6.9	Strike Slip	8.7
6	Kocaeli, Turkey	1999	Arcelik	7.5	Strike Slip	53.7
7	Loma Prieta	1989	Capitola	6.9	Strike Slip	9.8
8	Manjil, Iran	1990	Abbar	7.4	Strike Slip	40.4
9	Cape Mendocino	1992	Rio Dell Overpass	7.0	Thrust	22.7
10	Chi-Chi, Taiwan	1999	CHY101	7.6	Thrust	32
11	San Fernando	1971	LA - Hollywood Stor	6.6	Reverse	39.5
12	Friuli, Italy	1976	Tolmezzo	6.5	Thrust	20.2

CHAPTER-4

MODELLING OF URM INFILLS

4.1 GENERAL

Modeling of infills is a significant step for modelling of infilled frames as different models have been suggested by different researchers and no conclusion has been reached about which method to use. The existing models can be grouped into two categories – first one is Micro and other one is Macro model.

Micro models are centered around finite element representation of each infill panel. They are thus capable of accounting for the local infill–frame interaction and to understand and gain a much better perspective of the behavior in a much comprehensive manner. But the downside of this modelling is that they are computationally very costly.

Macro model are based on the diagonal compression action of an infill within a frame system. There is very high degree of in-homogeneousness and broadly diverse non-linear brittle behaviour of masonry units coupled with mortar resulting in time exhaustive and computationally difficult finite element problem in micro models, led to the acceptance of macro models.

4.2 DIAGONAL STRUT MODEL

Since Polyakov (1960) gave the idea of a strut model, many researchers have suggested many numerical models for the calculation of equivalent width of diagonal strut.

HOLMES (1961) [13]

$$w=d_z/3 \quad \ldots\ldots\ldots\ldots\ldots\ldots(3.3)$$

SMITH AND CARTER (1969) [13]

$$w=0.58\left(\frac{1}{H}\right)^{-0.445}(\lambda_h H')^{.335 d_z \left(\frac{1}{H}\right)^{.064}} \quad \ldots\ldots\ldots\ldots\ldots\ldots(3.4)$$

LIAW AND KWAN (1984) [8]

$$w=\frac{.95\ H\cos(\theta)}{\sqrt[2]{\lambda H}} \quad \text{...(3.5)}$$

$$\lambda = \sqrt[4]{\frac{Ef\ tsin(2\theta)}{4\ Em\ Ic\ H}} \quad \text{...(3.6)}$$

DECANINI AND FANTIN (1986) [2]

UNCRACKED MASONRY

$$w=(\frac{0.748}{\lambda_h}+.085)d_z \quad \text{...(3.7)}$$

CRACKED MASONRY

$$w=(\frac{0.707}{\lambda_h}+.01)d_z \quad \text{..(3.8)}$$

PAULEY AND PRIESTLEY (1992) [9]

$$w= d_z/4 \quad \text{...(3.9)}$$

FEMA 356 [4]

$$w=0.175d_z(\lambda H')^{-0.4} \quad \text{...(3.10)}$$

DRYSDALE, HAMID AND BAKER (1994) [3]

$$w=0.5\sqrt{{\alpha_h}^2+\alpha_l^2} \quad \text{where}$$

$$\alpha_h = \pi/2\sqrt[4]{\frac{E_f I_c H}{2E_m tsin(2\theta)}}$$

$$\alpha_L = \pi\sqrt[4]{\frac{E_f I_b L}{E_m t\ sin(2\theta)}} \quad \text{..............................(3.11)}$$

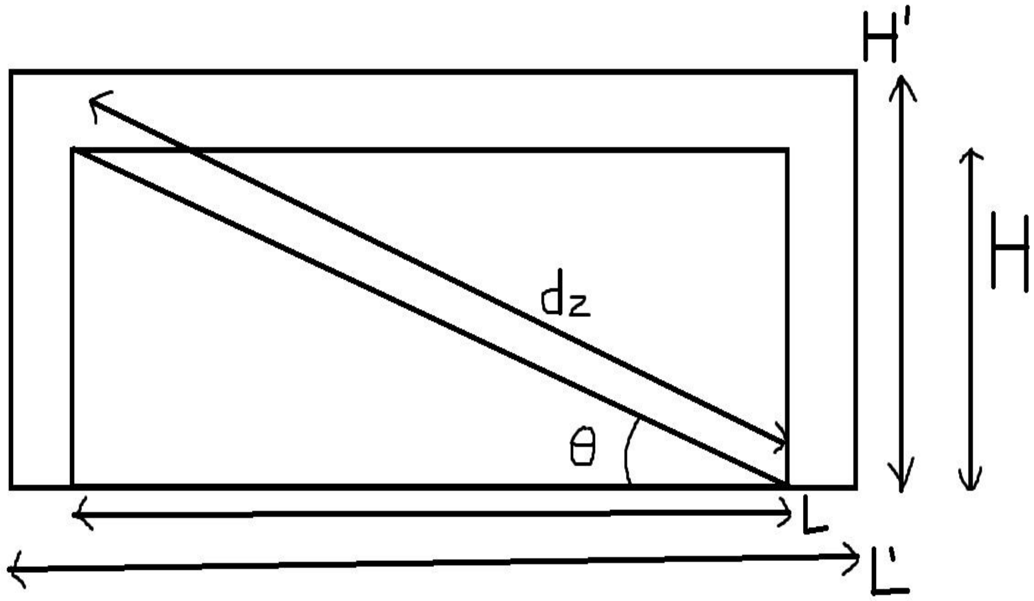


FIG. 4.1 Descriptions of equivalent Strut model [13]

where

E_f =Modulus of elasticity of frame

E_m=Modulus of elasticity of masonry

I_c=Moment of inertia of column

I_b= Moment of inertia of beam

t=Thickness of infill

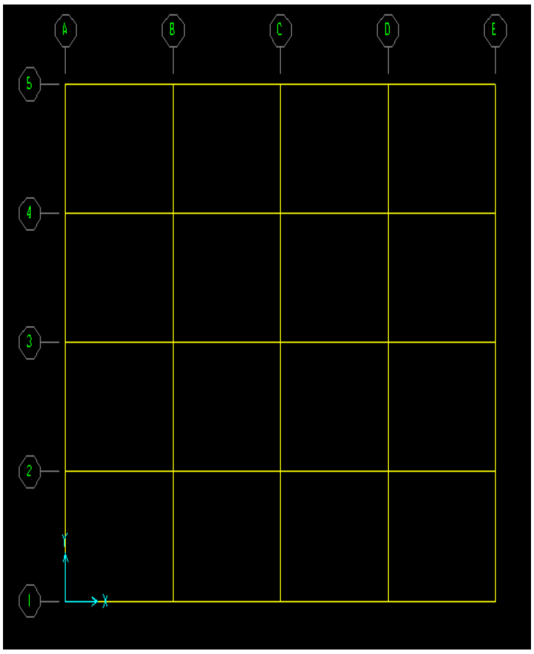
4.3 DESCRIPTION OF 4 STOREY BUILDING

In the present report, frame members are modeled as 3D frame elements with rigid end connection, floors as rigid diaphragm and infill as equivalent diagonal compression strut.

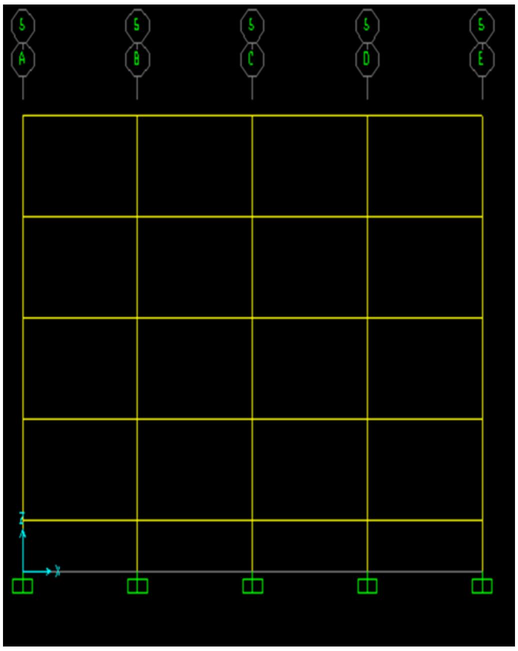
TABLE 4.1. Building Description

No. of storeys	4
Plan dimension	20*16m
Zone	IV
Soil type	Medium
Storey height	3m
Poisson’s ratio of masonry	0.15
Response reduction factor	5
Unit weight of concrete	25 kN/m^3
Unit weight of masonry	20 kN/m^3
Live load on floor	2.5 kN/m^2
Floor finish load	1.5 kN/m^2
Roof treatment	1.5 kN/m^2
Live roof	1.25 kN/m^2
Column size	400*400mm
Beam size	400*350mm
Thickness of infill	150mm
Foundation depth	1.5m

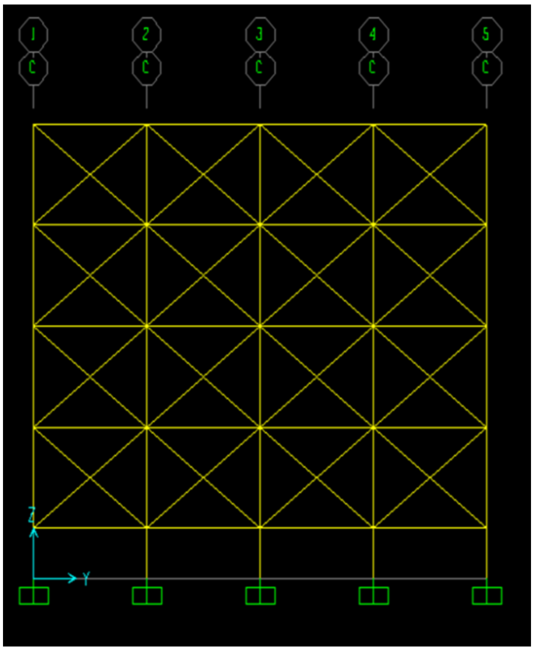
The building has been designed as SMRF for gravity as well as earthquake loading. The building is having 4 bays in both X and Y direction.



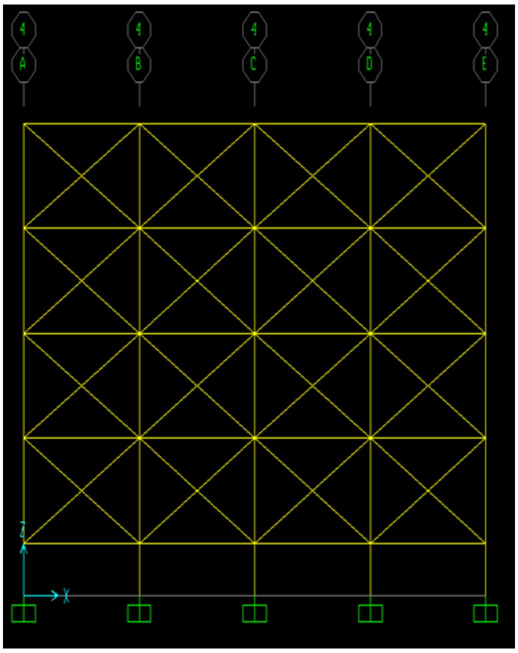
(a) Plan of building



(b)Elevation of bare frame



(c)Elevation(Y-Z) of infilled frame



(d) Elevation(X-Z) of infilled frame

FIG.4.2 Plan and elevation of 4 storey building

TABLE 4.2 Equivalent strut width by different methods

Equivalent width	Wx(m)	Wy(m)
Holmes	1.760	1.480
Liaw and Kwan	1.254	1.156
Pauley and Priestley	1.320	1.110
Durrani &Luo	1.107	1.067
FEMA 356	0.600	0.500
Drysdale,Hamid & Baker	1.315	1.247

TABLE 4.2 Time period(s) corresponding to different strut width

Mode	Holmes	Liaw	Pauley	Durrani	FEMA	Drysdale	IS code
1	0.282	0.297	0.299	0.314	0.370	0.292	0.303
2	0.273	0.292	0.289	0.302	0.353	0.289	0.271
3	0.243	0.260	0.260	0.271	0.322	0.256	
4	0.089	0.095	0.096	0.099	0.122	0.092	
5	0.083	0.090	0.089	0.097	0.114	0.089	
6	0.070	0.077	0.077	0.083	0.104	0.076	
7	0.069	0.071	0.071	0.072	0.072	0.071	
8	0.058	0.062	0.061	0.065	0.069	0.061	
9	0.055	0.060	0.054	0.062	0.067	0.059	
10	0.051	0.054	0.053	0.057	0.065	0.053	
11	0.050	0.051	0.051	0.056	0.064	0.053	
12	0.047	0.050	0.049	0.056	0.063	0.050	

After computing the equivalent strut width of the infill using Drysdale method and then carrying out free vibration analysis of the model, the fundamental time period is very much comparable with IS code method. Also the Drysdale method takes into account the effect of column and beam interaction in calculating the equivalent strut width. Hence Drysdale method will be used for further work in this dissertation.

CHAPTER-5

RESULTS AND DISCUSSIONS

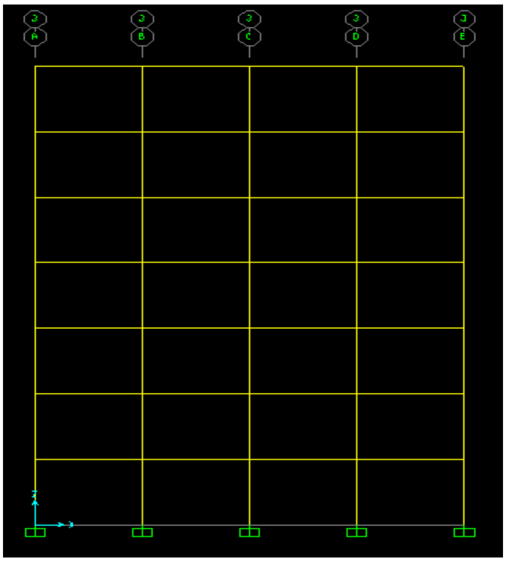
5.1 DESCRIPTION OF 7 STOREY BUILDING

In the present report, frame members are modeled as 3D frame elements with rigid end connection, floors as rigid diaphragm and infill as equivalent diagonal compression strut. Equivalent width of strut is calculated using Drysdale method [3].

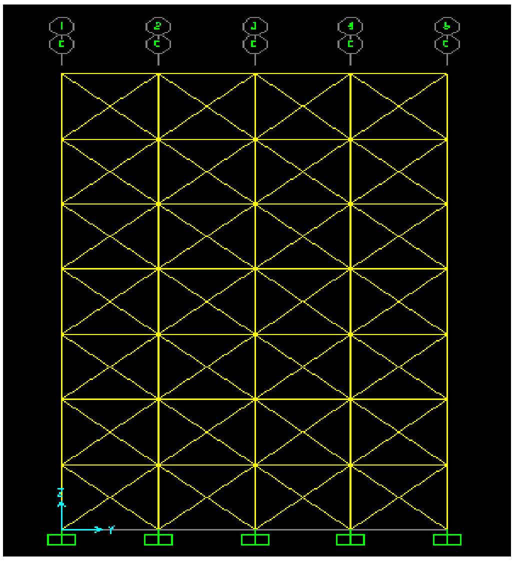
TABLE 5.1 Building Description

No. of storeys	7
Plan dimension	20*16m
Zone	IV
Soil type	Medium
Storey height	3.7m
Poisson ratio of masonry	0.15
Response reduction factor	5
Unit weight of concrete	25kN/m^3
Unit weight of masonry	20 kN/m^3
Live load on floor	2.5 kN/m^2
Floor finish load	1.5 kN/m^2
Roof treatment	1.5 kN/m^2
Live roof	1.25 kN/m^2
Column size	400*400 mm
Beam size	400*350 mm
Thickness of infill	150 mm
Foundation depth	0 m

The plan of the building is same as that of 4 storey building.



(a)Elevation of bare frame



(b)Elevation of infilled frame

FIG.5.1 Elevation of bare and infilled 7 storey building

TABLE 5.2 Time Period(s) of Infilled frame buildings

Mode	4 Storey	7 Storey
1	0.2922	0.4330
2	0.2890	0.3930
3	0.2568	0.3220
4	0.0920	0.1460
5	0.0890	0.1386
6	0.0760	0.1150
7	0.0716	0.1090
8	0.0616	0.0849
9	0.0598	0.0840
10	0.0538	0.0730
11	0.0535	0.0720
12	0.0500	0.0670

TABLE 5.3 Displacement(m) of 4 storey building under earthquake in X direction

Storey height	Bare frame	Infilled frame
0	0	0
1.5	0.0021	0.0007
4.5	0.0137	0.0011
7.5	0.0258	0.0013
10.5	0.0349	0.0014
13.5	0.0399	0.0015

TABLE 5.4 Displacement (m) of 4 storey building under earthquake in Y direction

Storey height	Bare frame	Infilled frame
0	0	0
1.5	0.0019	0.0007
4.5	0.0122	0.0011
7.5	0.0228	0.0013
10.5	0.0305	0.0015
13.5	0.0347	0.0016

TABLE 5.5 Displacement(m) of 7 storey building under earthquake in X direction

Storey height	Bare frame	Infilled frame
0	0	0
3.7	0.0156	0.0006
7.4	0.0417	0.0013
11.1	0.0670	0.0019
14.8	0.0889	0.0025
18.5	0.1062	0.0031
22.2	0.1182	0.0035
25.9	0.1250	0.0038

TABLE 5.6 Displacement(m) of 7 storey building under earthquake in Y direction

Storey height	Bare frame	Infilled frame
0	0	0
3.7	0.0142	0.0006
7.4	0.0372	0.0013
11.1	0.0592	0.021
14.8	0.0779	0.0028
18.5	0.0928	0.0035
22.2	0.103	0.004
25.9	0.1087	0.0045

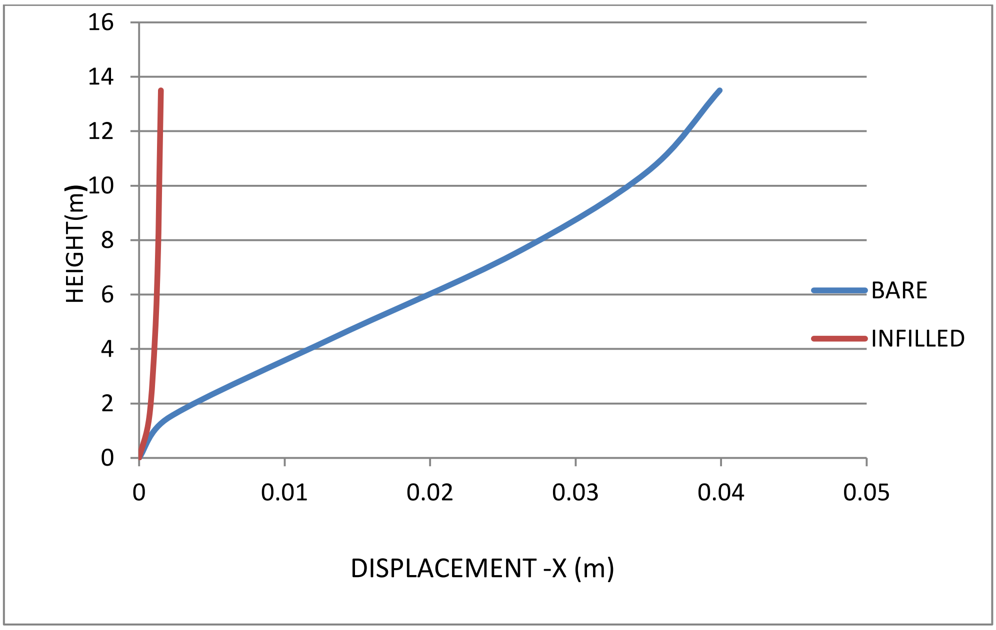


FIG.5.2 Displacement of 4 storey building under earthquake in X direction

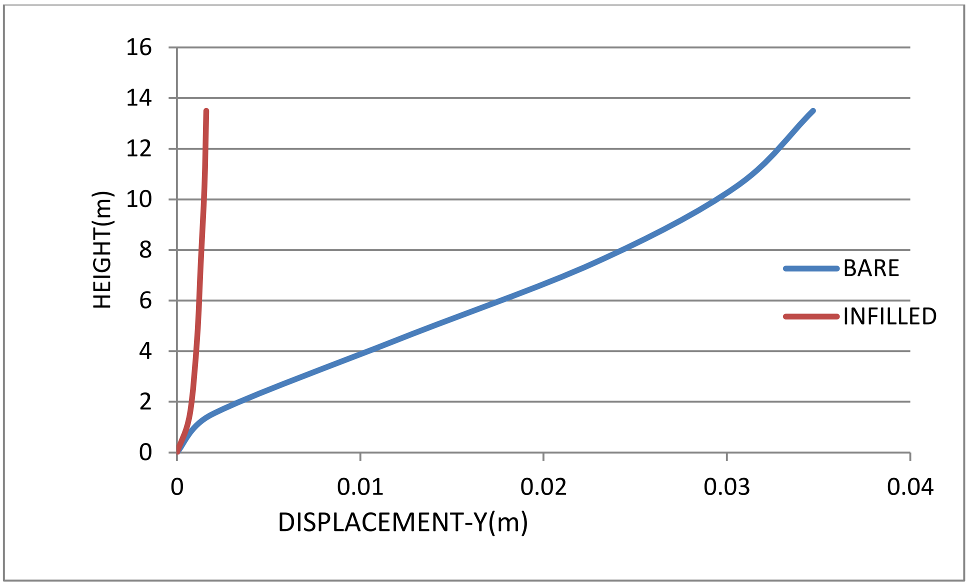


FIG5.3 Displacement of 4 storey building under earthquake in Y direction

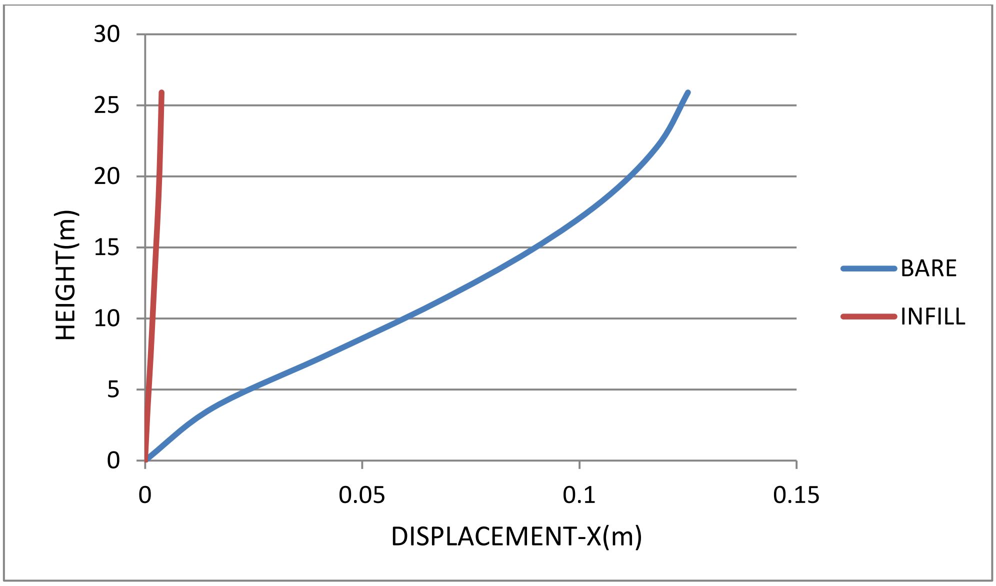


FIG.5.4 Displacement of 7 storey building under earthquake in X direction

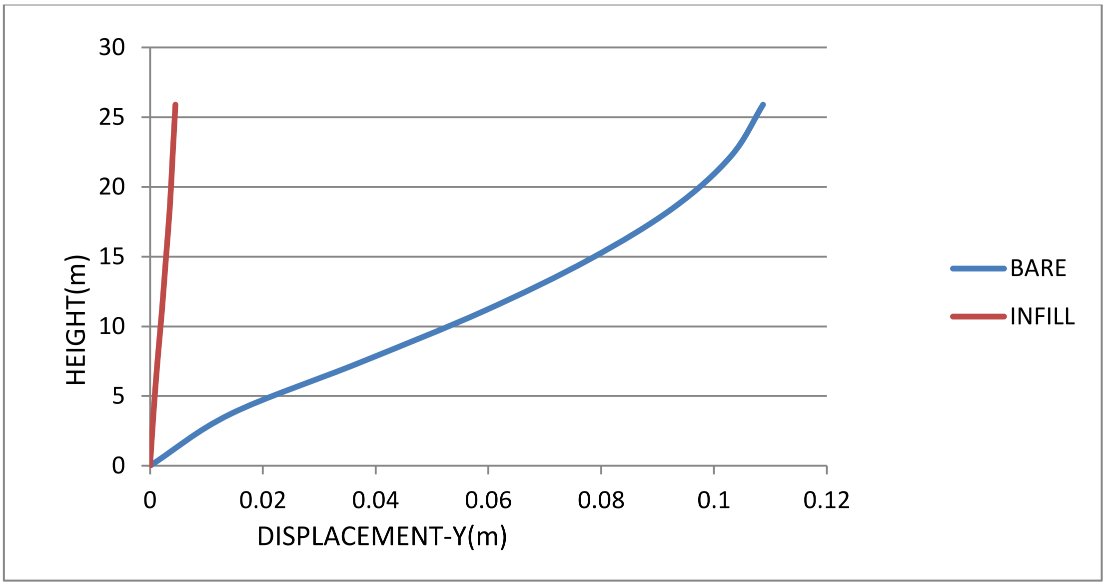


FIG.5.5 Displacement of 7 storey building under earthquake in Y direction

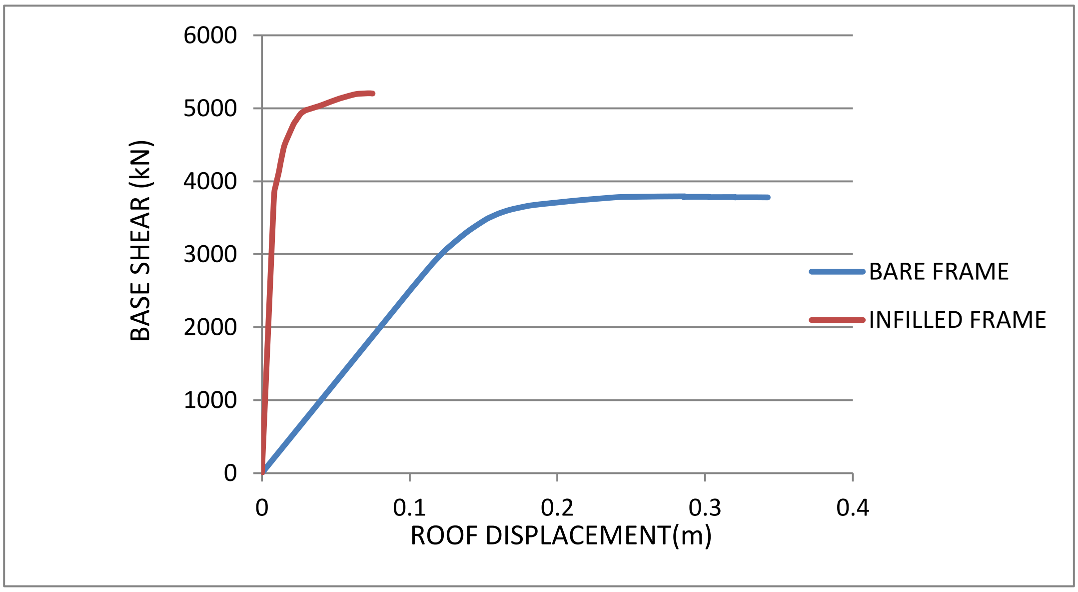


FIG.5.6 Pushover analysis of 4 storey building in X direction

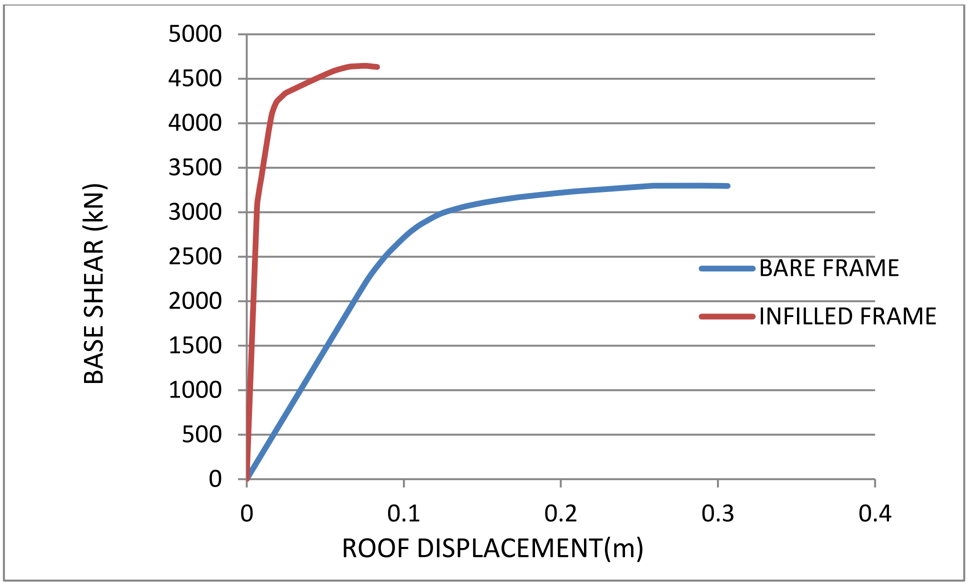


FIG.5.7 Pushover analysis of 4 storey building in Y direction

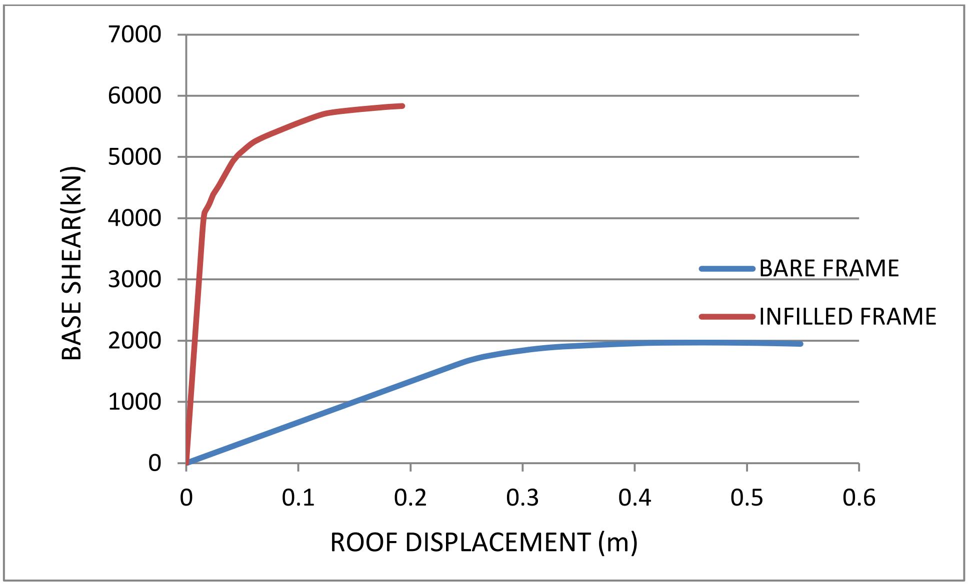


FIG.5.8 Pushover analysis of 7 storey building in X direction

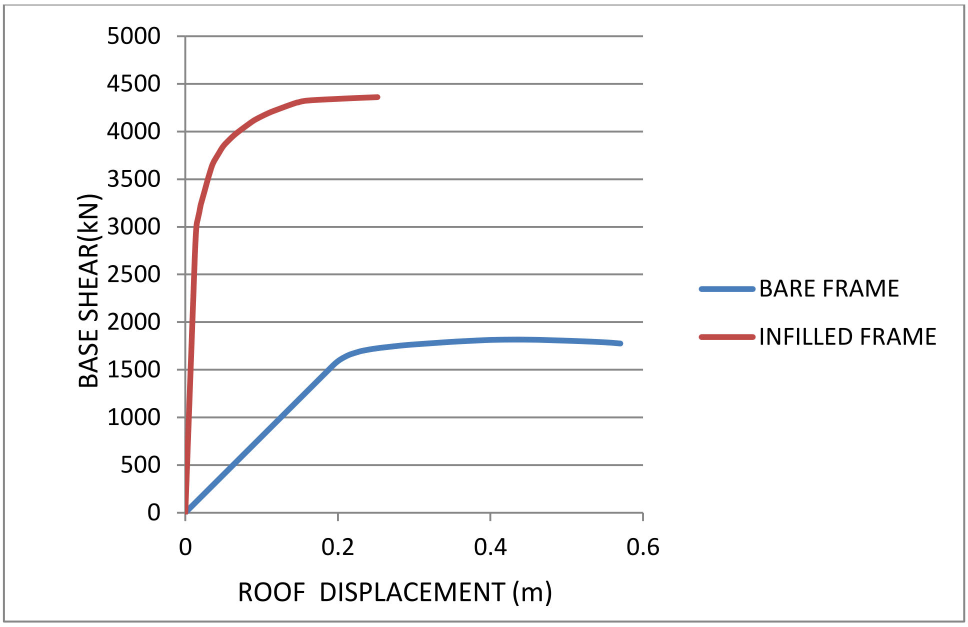
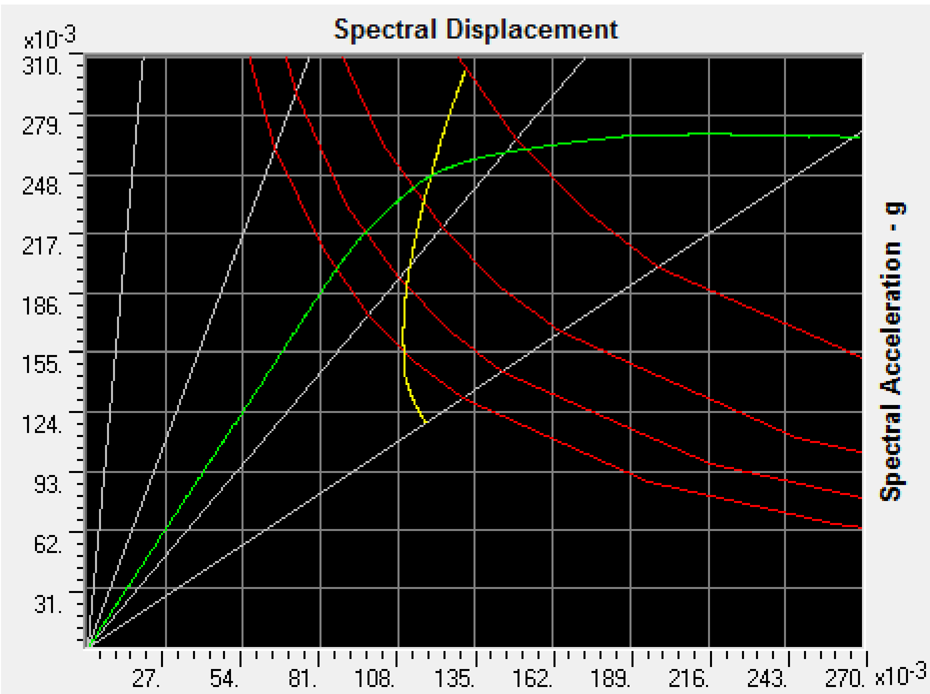


FIG.5.9 Pushover analysis of 7 storey building in Y direction

The next 8 graphs show the performance point obtained in SAP2000 for the different building types

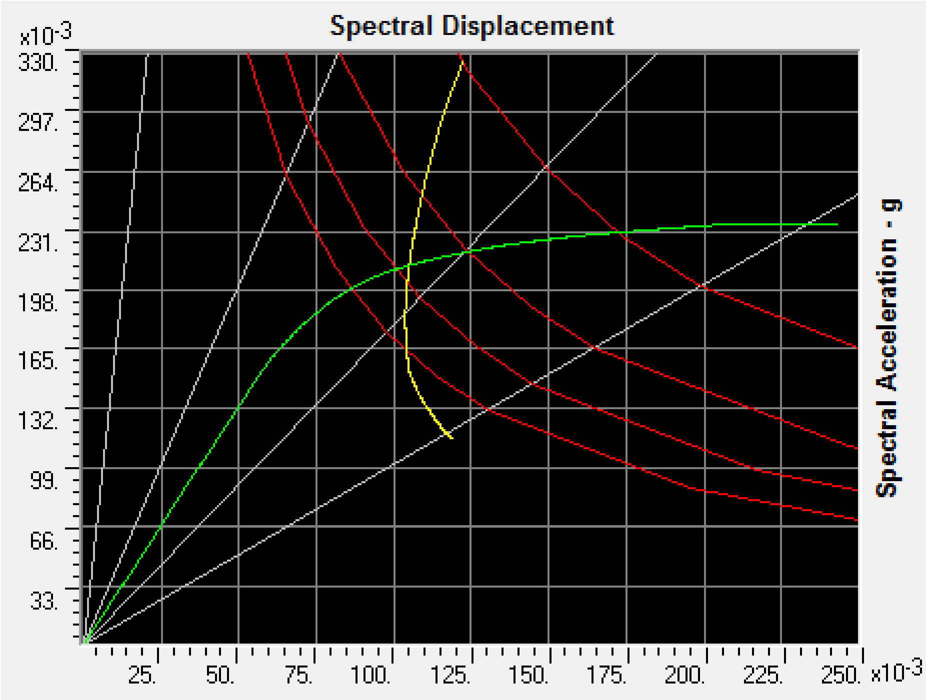
Spectral displacement (m) vs Spectral acceleration g (m/s^2)



S_a=.248 g m/s^2

S_d=.119m

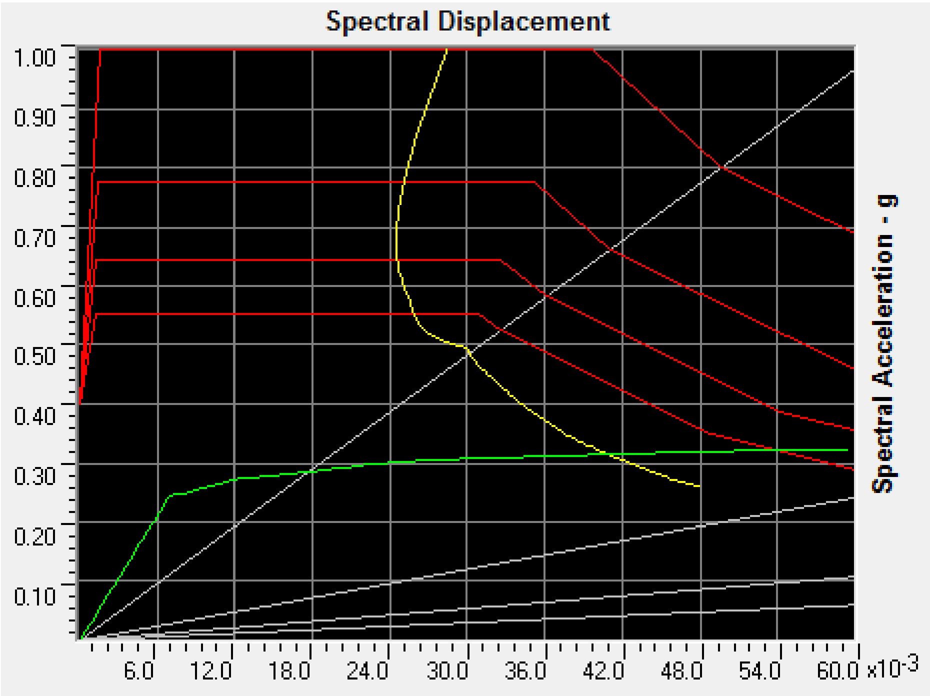
FIG.5.10 ATC 40 Capacity spectrum showing performance point of 4 storey bare building in X direction



S_a=.212 g m/s^2

S_d=.105 m

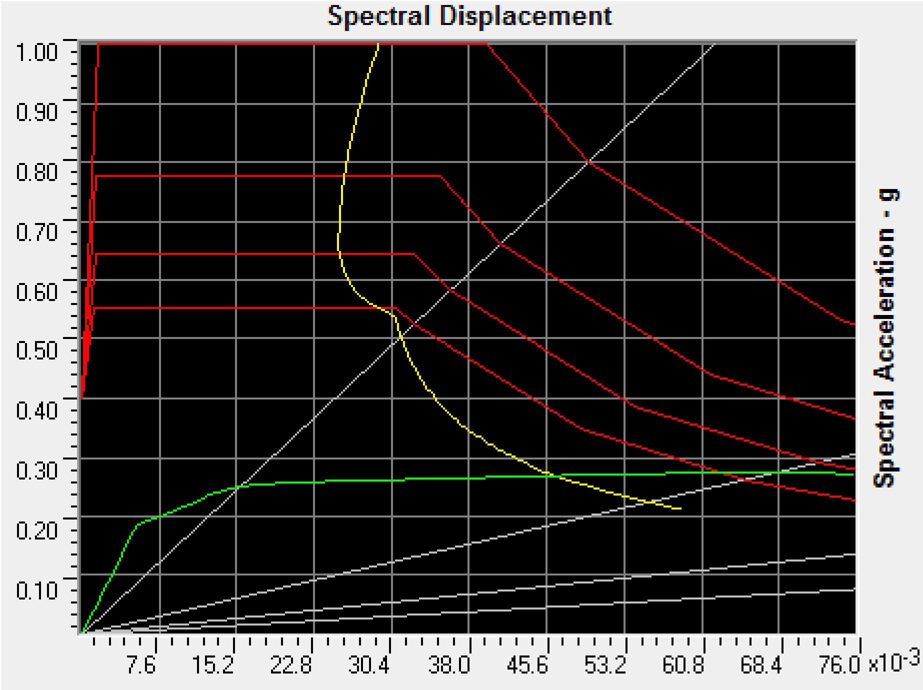
FIG.5.11 ATC 40 Capacity spectrum showing performance point of 4 storey bare building in Y direction



S_a=.315g m/s^2

S_d=.041 m

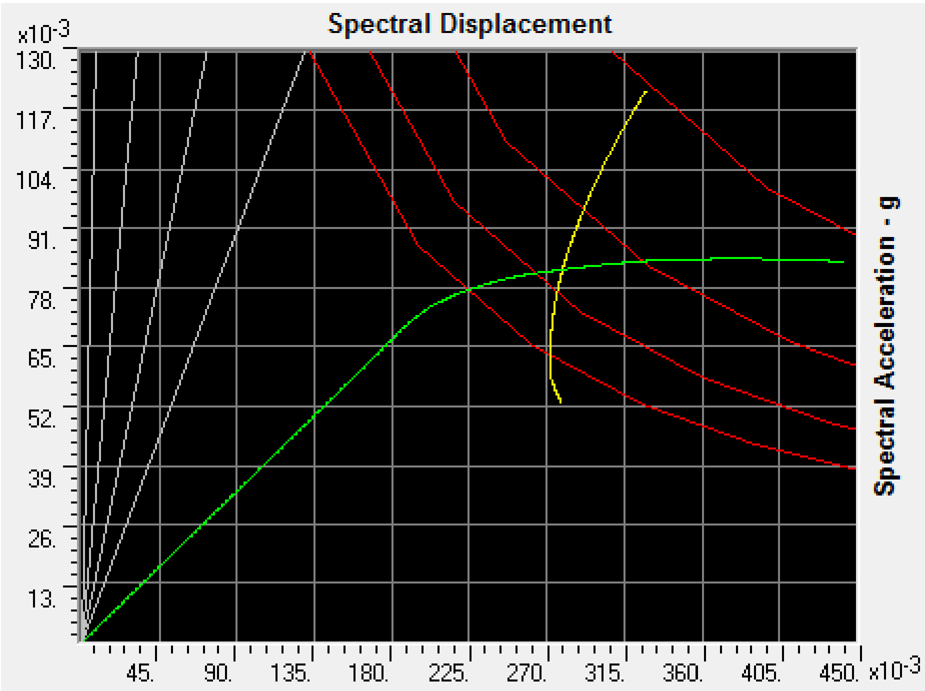
FIG.5.12 ATC 40 Capacity spectrum showing performance point of 4 storey infilled building in X direction



S_a=.269 g m/s^2

S_d=.046 m

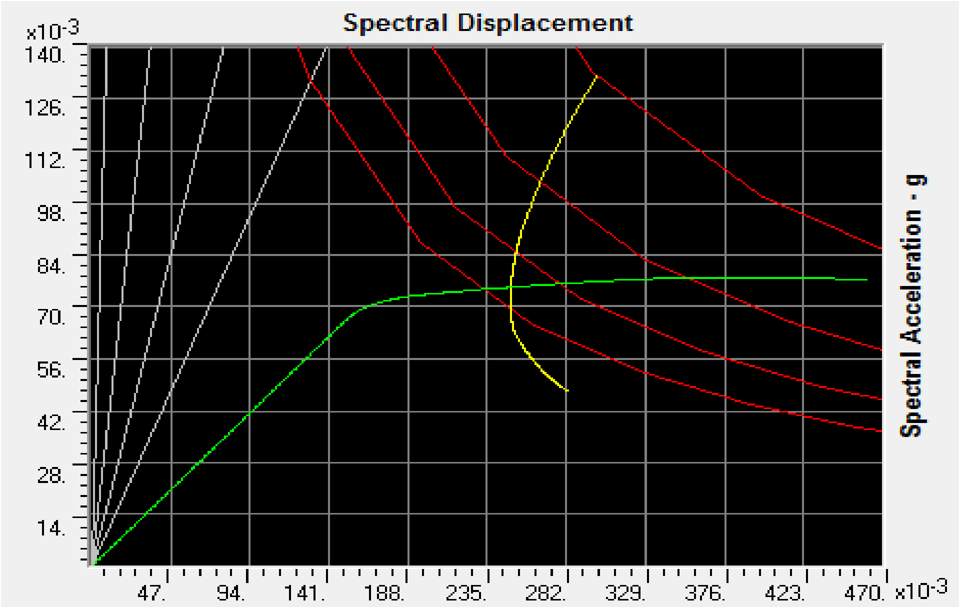
FIG. 5.13 ATC 40 Capacity spectrum showing performance point of 4 storey infilled building in Y direction



S_a=.082 g m/s^2

S_d=.279 m

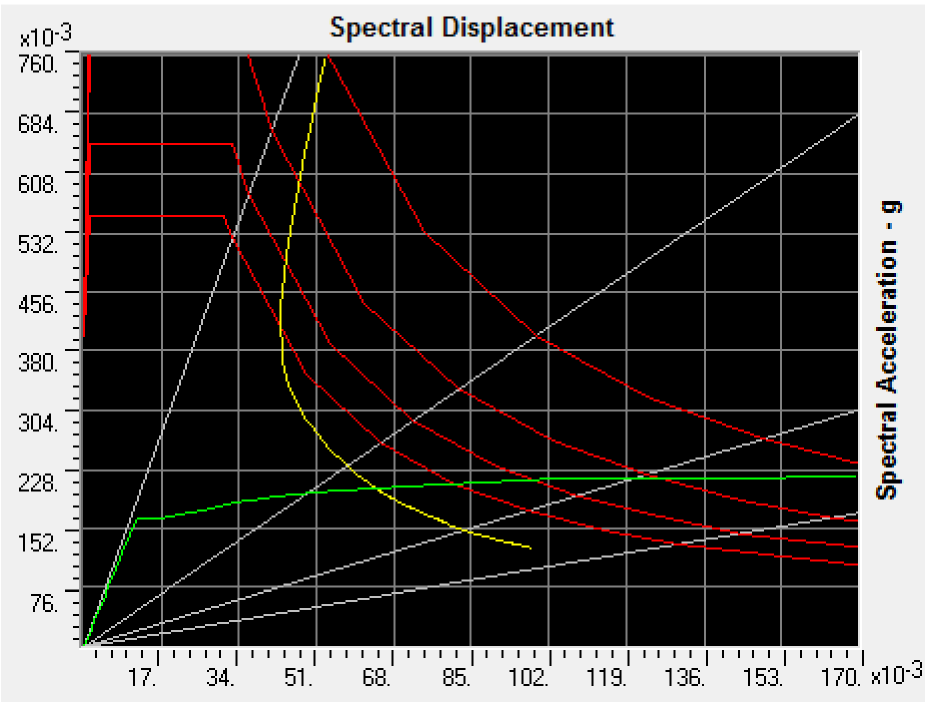
FIG.5.14 ATC 40 Capacity spectrum showing performance point of 7 storey bare building in X direction



S_a=.075 g m/s^2

S_d=.249m

FIG. 5.15 ATC 40 Capacity spectrum showing performance point of 7 storey bare building in Y direction



S_a=.204 g m/s^2

S_d=.064 m

FIG. 5.16 ATC 40 Capacity spectrum showing performance point of 7 storey infilled building in X direction

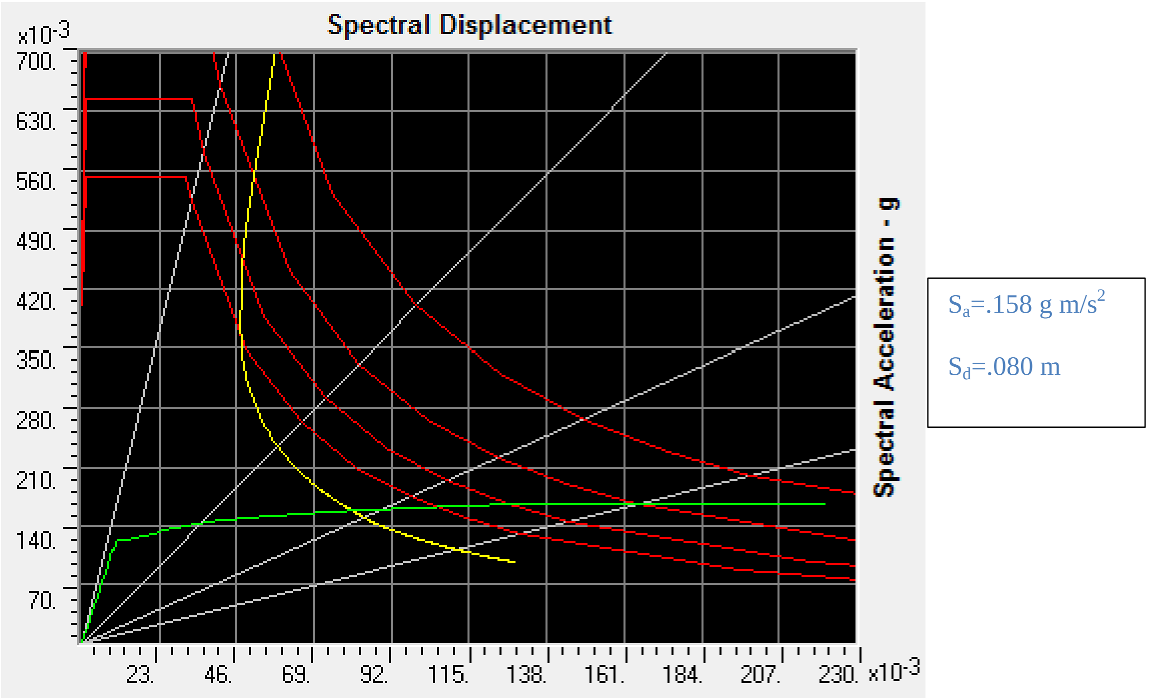


FIG. 5.17 ATC 40 Capacity spectrum showing performance point of 7 storey infilled building in Y direction.

TABLE 5.7 Performance point of buildings

Performance Point	**Sa-g (m/s^2)**	**Sd (m)**
4 Storey bare frame-X	.248	.119
4 Storey bare frame-Y	.212	.105
4 Storey infilled frame-X	.315	.041
4 Storey infilled frame-Y	.269	.046
7 Storey bare frame-X	.082	.279
7 Storey bare frame-Y	.075	.249
7 Storey infilled frame-X	.204	.064
7 Storey infilled frame-Y	.158	.080

Performance point is defined as the point of intersection of capacity curve and the damped demand curve. At this point, damping is same for both demand and capacity curves.

5.3 INCREMENTAL DYNAMIC ANALYSIS

It is a non-linear dynamic analysis procedure in which the building is subjected to various scaled time histories so that the structure can move from elastic to inelastic state and ultimately collapse. Original ground motion may not be sufficient to make the stucture collapse so we have to scale the ground motion. It will give the demand parameter which is in the form of inter storey drift which will then be used for defining various damage state of the structure. 12 different ground motion are selected and they are scaled until the structure collapses.

Inter storey drift(%) for the infilled frame has been obtained from the work done by Kalman and Singmund [15]. Using the various states defined by the authors the damage state of the infilled frame has been calculated.

TABLE 5.8 Limits for different damage states in term of inter-storey drift(%) for bare frame

State of Damage	Interstorey Drift Ratio(%)
Slight Damage	IDR≤0.21
Moderate Damage	0.21<IDR≤0.64
Extensive Damage	0.64<IDR≤1.60
Near Collapse	IDR>1.60

TABLE 5.10 Limits for different damage states in terms of inter-storey drift (%) for infilled frame [15]

State of Damage	Interstorey Drift Ratio(%)
Slight Damage	IDR≤0.1
Moderate Damage	0.1<IDR≤0.3
Extensive Damage	0.3<IDR≤0.75
Near Collapse	IDR>0.75

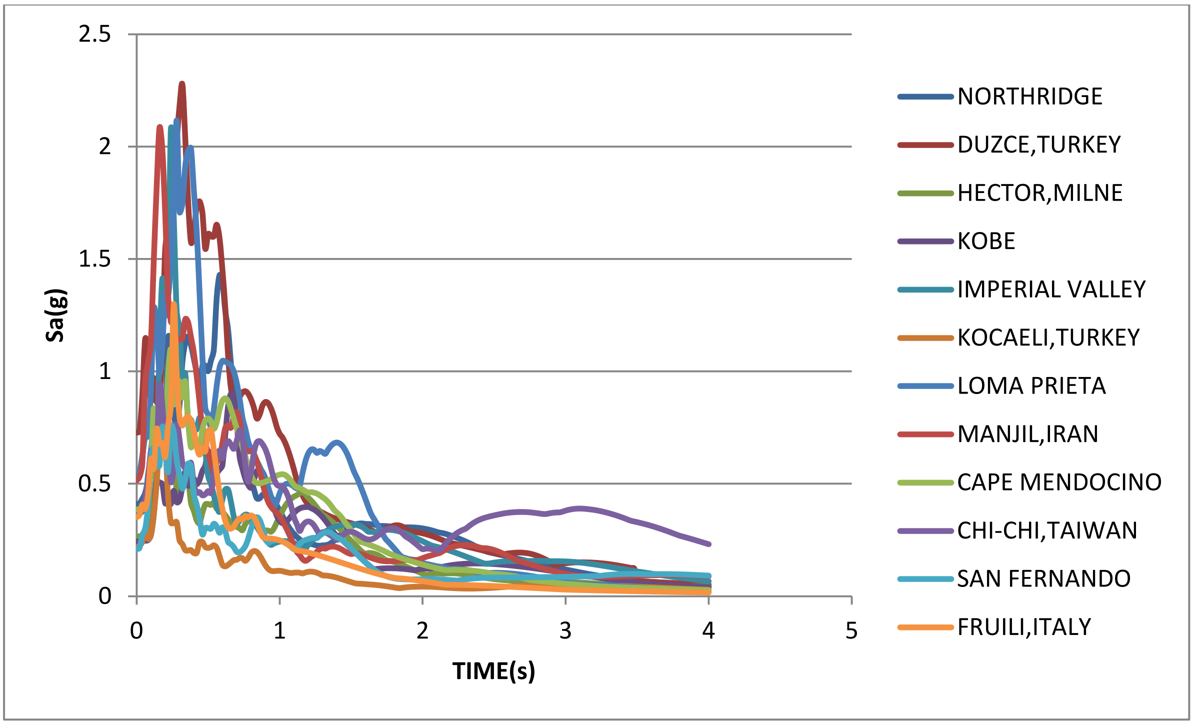


FIG. 5.18 Response Spectra of various ground motions used for IDA

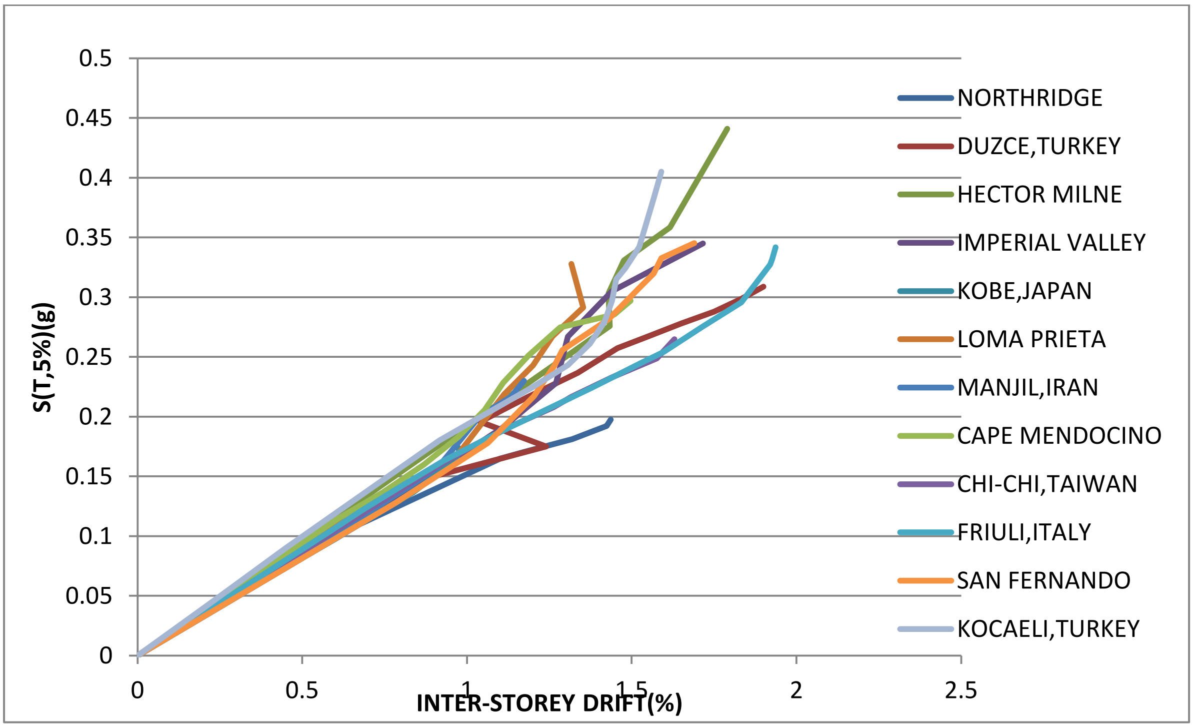


FIG. 5.19 Peak inter-storey drift(%) v/s Spectral acceleration (Fundamental Mode) for Bare frame

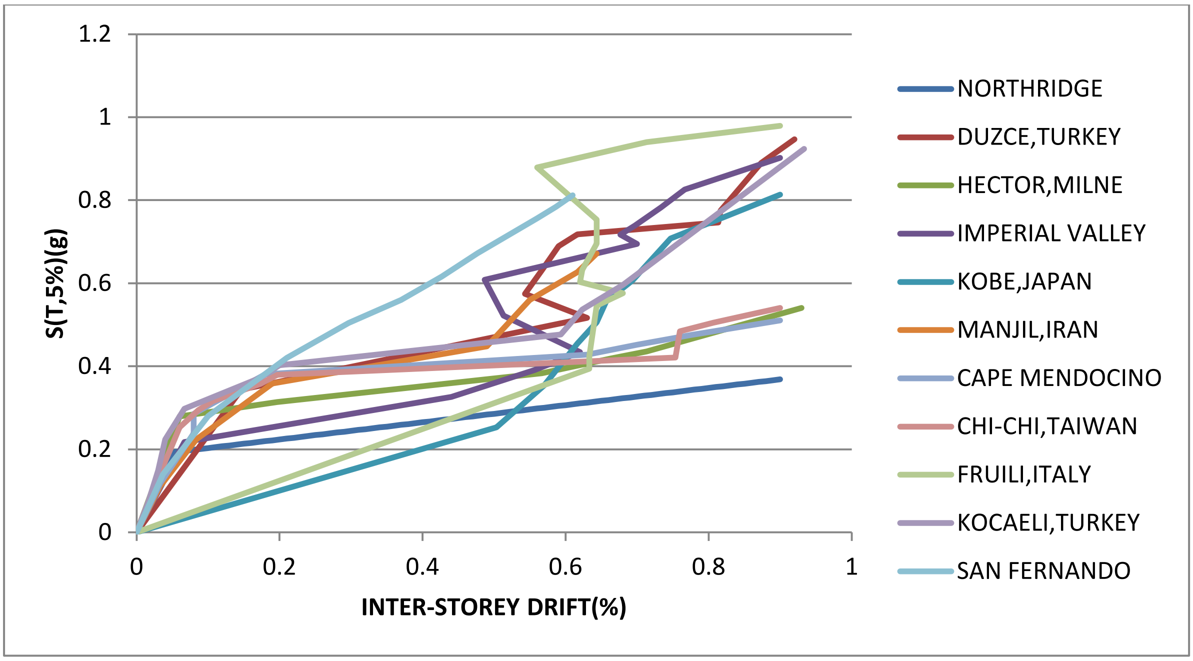


FIG.5.20 Peak inter-storey drift(%) v/s Spectral acceleration(Fundamental Mode) for infilled frame

TABLE 5.10 Beta values for Bare frame

Parameter	Slight Damage	Moderate Damage	Extensive Damage	Collapse
Median	0.0188	0.07602	0.2021	0.3399
β_m	0.3000	0.3000	0.3000	0.3000
β_c	0.5654	0.2930	0.3768	0.3057
β	0.6393	0.4193	0.4816	0.42835

TABLE 5.11 Beta values for infilled frame

Parameter	Slight Damage	Moderate Damage	Extensive Damage	Collapse
Median	0.1393	0.3395	0.4545	0.8047
β_m	0.3000	0.3000	0.3000	0.3000
β_c	0.6243	0.2935	0.3529	0.3157
β	0.6926	0.4197	0.4632	0.4355

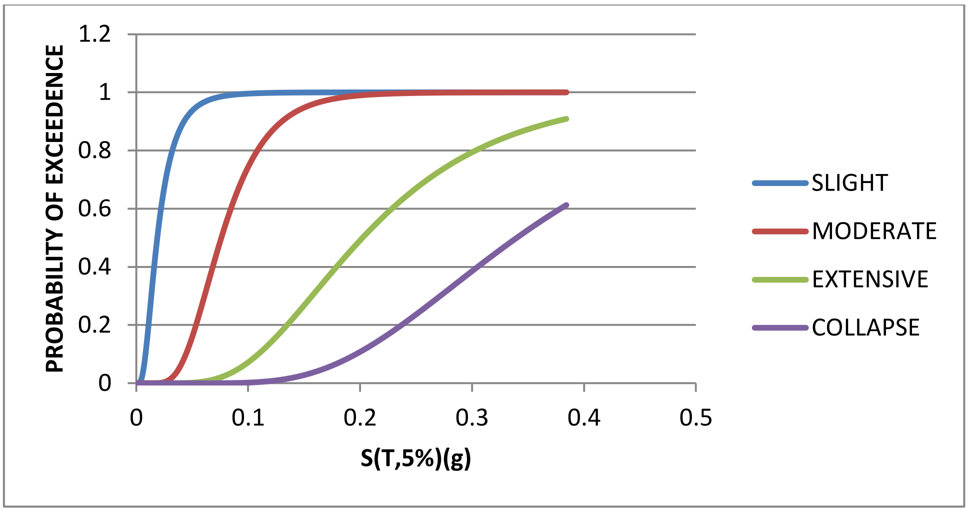


FIG. 5.21 Fragility curve for bare frame showing probability of exceedance v/s spectral acceleration

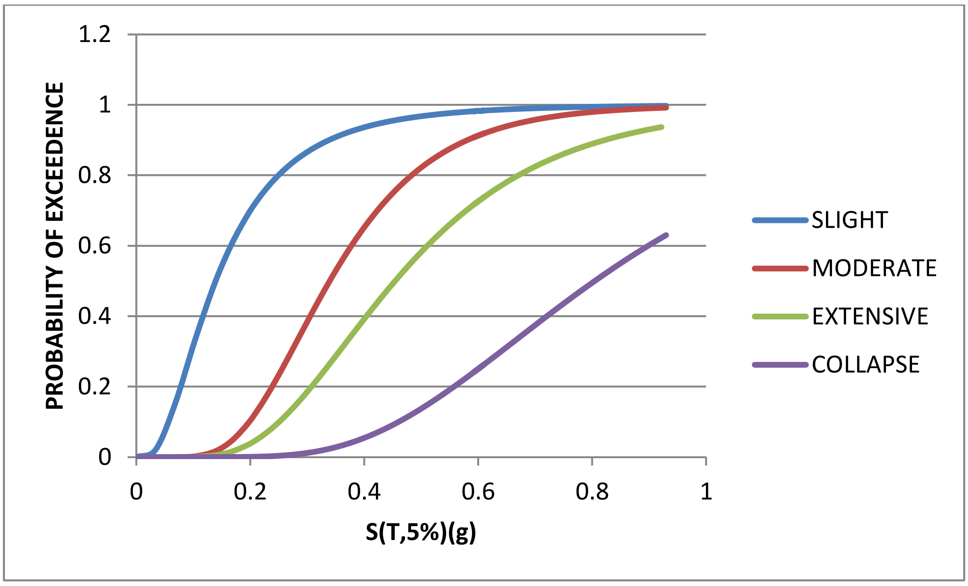


FIG. 5.22 Fragility curve for infilled frame showing probability of exceedance v/s spectral acceleration

CHAPTER-6

SUMMARY AND CONCLUSIONS

6.1 SUMMARY

In the present report, firstly different models are compared to calculate equivalent strut width and time period. Drysdale method was chosen for further analysis of the building. Two building were chosen of same plan, one 4 storey and other 7 storey. Pushover analysis was performed for both buildings for both bare and infilled frame building. Performance points of the buildings were calculated. After that IDA was performed for 4-storey building. Finally fragility curves have been plotted.

6.2 CONCLUSIONS

Based on the above study, following conclusions can be drawn;

1. Addition of infills to building increases its strength and stiffness but the inelastic deformation capacity of the building reduces.
2. Holmes method gives the highest strut width while FEMA 356 gave the lowest value, meaning Holmes method gives the stiffer model while FEMA 356 gives the least stiffer model.
3. In in-filled frame building inter storey drift decreases due to increase in stiffness.
4. Structure can be subjected to higher base shear as infill helps in energy dissipation.
5. In infilled frame building, plastic hinges first form in strut, then in beams and then in columns.
6. The formation of hinges in struts is first in ground storey, and then it propagates to upper storeys.
7. If the real ground motion is not sufficient to cause the collapse of the building then we need to scale up the ground motion and vice-versa.
8. In order to have the same inter-storey drift, spectral acceleration will be more in infilled frame than in bare frame.

6.3 SCOPE FOR FUTURE WORK

The modeling of infill is still a process on which a consensus has not been reached till now. Also the complex behavior of infill makes it a tough task to select the modelling basis. The behavior of infills is greatly modified due to the presence of openings which provides a greater scope for research work.

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