EFFECT OF INFILL WALLS IN STRUCTURAL RESPONSE OF RC BUILDINGS

BY

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Contents

Introduction

- In the last decade the strong scientific and legislative introduced a revolution in design philosophy with respect to Earthquake

- Many existing buildings in the World, designed and constructed until the late 1970's without considering adequate earthquake provisions, constitute a significant potential risk (economical and social) for our society

Assigned different tasks for structure engineer

- Vulnerability and seismic behavior of RC Buildings (Lesson from recent EQ's)

- Introducing of Tools and strategies (Monitoring and Evaluation)

- Introducing of some Experimental work on the assessment of Existing RC structures and Structural Components

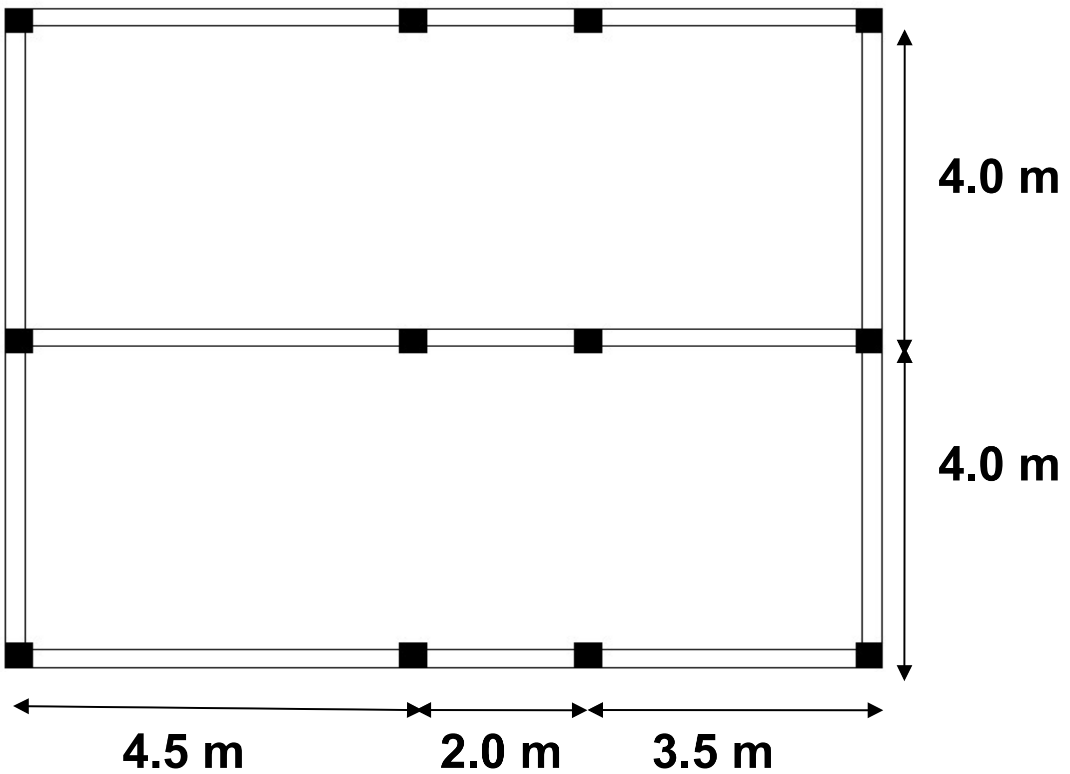
- Discuss the potentialities and limitations of refined numerical models in the seismic response representation of old RC structures

- Introducing of possible seismic retrofitting solutions

- But in spite of introduction of Modern codes, modern tools and techniques infill walls are still consider as non structural element
- The response of reinforced concrete buildings to earthquake loads can be substantially affected by the influence of infill walls
- Masonry infill in reinforced concrete buildings causes several undesirable effects under seismic loading: short-column effect, soft-storey effect, torsion, and out-of-plane collapse. Hence, seismic codes tend to discourage such constructions in high seismic regions
- It is inadequate to assume that masonry infill panels are always beneficial in terms of structural response
- The contributions of infills to the building's seismic response can be positive or negative, depending on a series of phenomena and parameters such as, for example, relative stiffness and strength between the frames and the masonry walls
- In the study reported here, elaborates the effect of infill walls in the seismic response of reinforced concrete (RC) buildings

Introduction Of The Building (System Level)

Plan Dimension of the Building Under Consideration

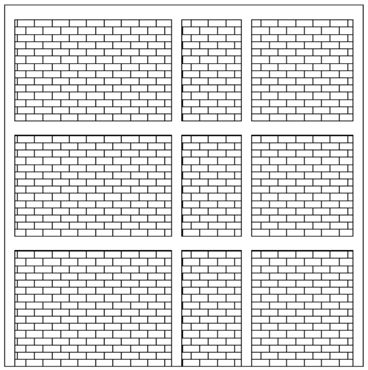


2-D Model (System Level)

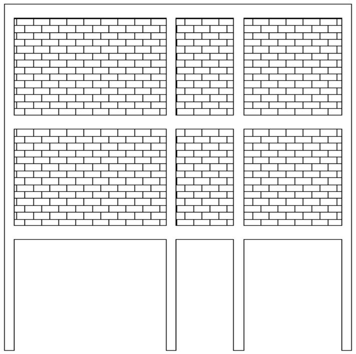
Potenza Italy, The PGA is 0.3g

To investigate the influence of Infill walls we have 3 Types of models

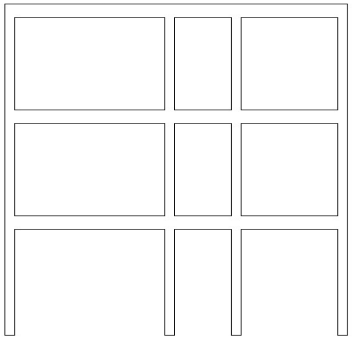
1- Full infill walls



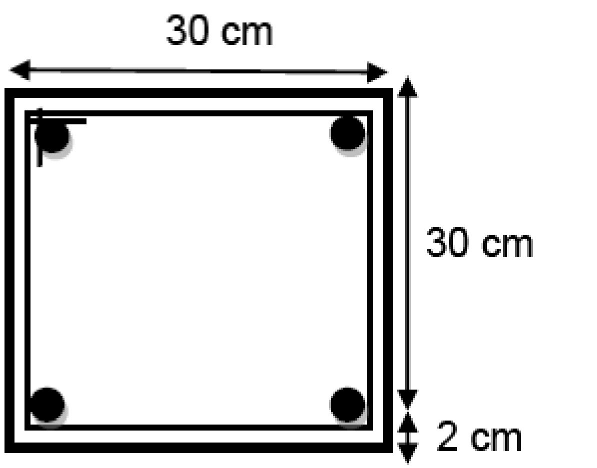
2- Soft-story



3- Bare Frame

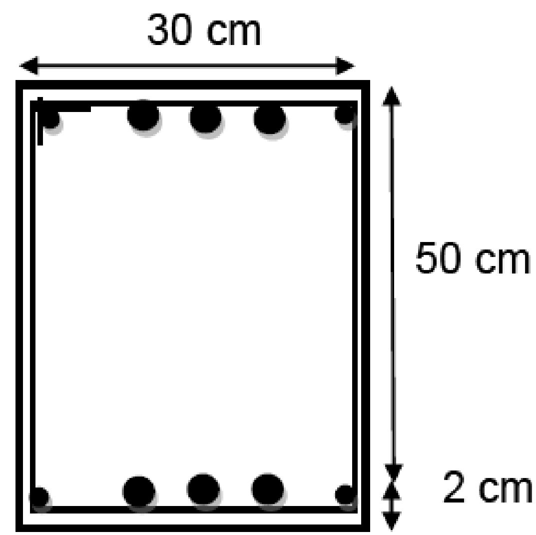


Columns' Cross-Section



ϕ 18 mm longitudinal Bars
ϕ 6 mm Stirrups @ 15 cm c/c

Beams' Cross-Section



ϕ 12 mm and ϕ 18 mm main Bars
ϕ 6 mm Stirrups @ 15 cm c/c

Modeling (Component Level)

- Expected Elastic Modulus for Fair frame Material is;
- Efe = 4206.1 Ksi (29 Gpa)

- Concrete Compression Strength is;

  fc' = 2250 Psi (15.52 Mpa)

- Steel used in Columns and Beams are smooth Bars having yield strength is;

  f_y = 20300 Psi (139.96 Mpa)

According to FEMA-356, Chapter-6, Table 6-7 & 6-8 Different Limit states for the nonlinear hinge should be define to capture the nonlinear behavior of RC structure's components

Table 6-7 Use to define the Back Bone Curve's Properties and also to define the different limit state of the Beams

Table 6-8 Use to define the Back Bone Curve's Properties and also to define the different limit state of the Columns

Table 6-7 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

Conditions			Modeling Parameters[3]			Acceptance Criteria[3]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.010	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01

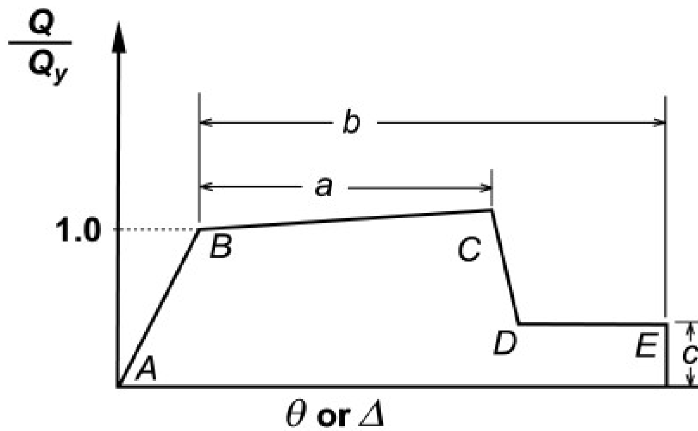
Table 6-8 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Columns

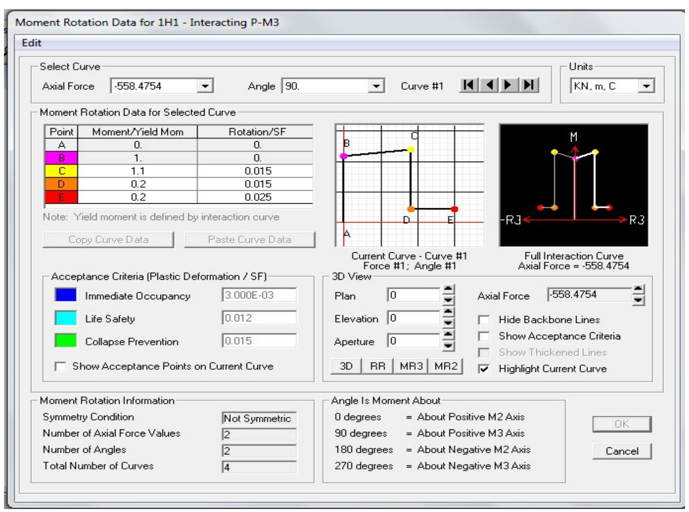
Conditions			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008

Table 6-7,8 of FEMA-356

Modeling (Component Level)

According to FEMA-356 Chapter-6 Table 6-7 & 6-8 Different Limit states for the nonlinear hinge for Beams and Columns should be Define in the computer Program





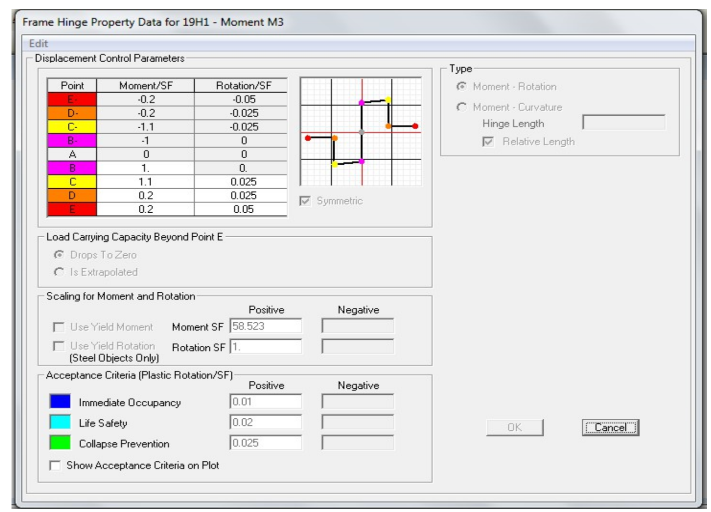


Table 6-7,8 of FEMA-356

FEMA-356 Single Strut Compression Model

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf}$$

$$A_e = ta$$

where:

$$\lambda_1 = \left[\frac{E_{me} t_{inf} \sin 2\theta}{4 E_{fe} I_{col} h_{inf}}\right]^{\frac{1}{4}}$$

and

$$V_{ine} = A_{ni} f_{vie}$$

A_{ni} **is the area of infill walls**

•Expected Elastic Modulus for Fair Masonry infill walls in Compression is equal to E_{me} = 550fm'=330 Ksi, where fm' is the compression strength of masonry equal to 600psi (4.1368 Mpa)

•Expected Shear Strength of Masonry is;
f_{vie} = 20 Psi (0.1378 Mpa)

Table 7-1 of FEMA-356

FEMA-356 Single Strut Compression Model

Compressive strength by ASCE-41/FEMA-356 Table 7-1	f'_m	600	Psi
Elastic of modulus in compression of infill	E_{me}	330	Ksi
Length of infill	L_{inf}	66.929	in
Thickness of infill	t_{inf}	9	in
Diagonal angle of infill	Θ	0.97	Radian
Elastic of modulus in compression of Frame	E_{fe}	3243.04	Ksi
Gross moment of inertia	I_g	1621.681013	in^4
Column moment of inertia	I_{col}	810.8405066	in^4
Height of infill	h_{inf}	98.425	in
Coefficient of infill	λ_1	0.040443751	
Height of column	h_{col}	118.11	in
I inclined length of infill	r_{inf}	119.0250884	in
Width of infill	a	0.093623056	
Width of infill	a	11.14349247	in
Area of infill strut	Ae	100.322	in2
Horizontal stiffness of infill strut	K_{inf}	278.0604752	Kip/in
Ratio	L_{inf}/h_{inf}	0.68	
β = Vfre/Vine	β	22.6566793	
Axial strength of infill i.e. strut	Pno	50.22145	Kip

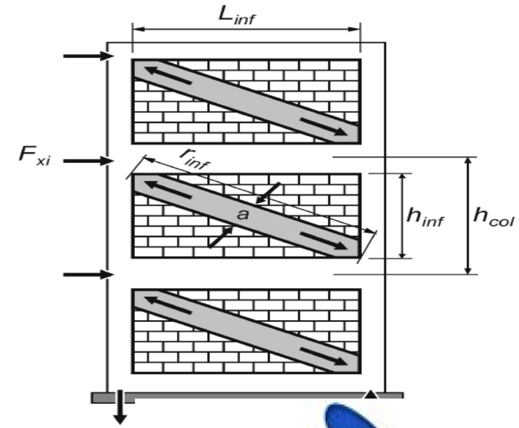
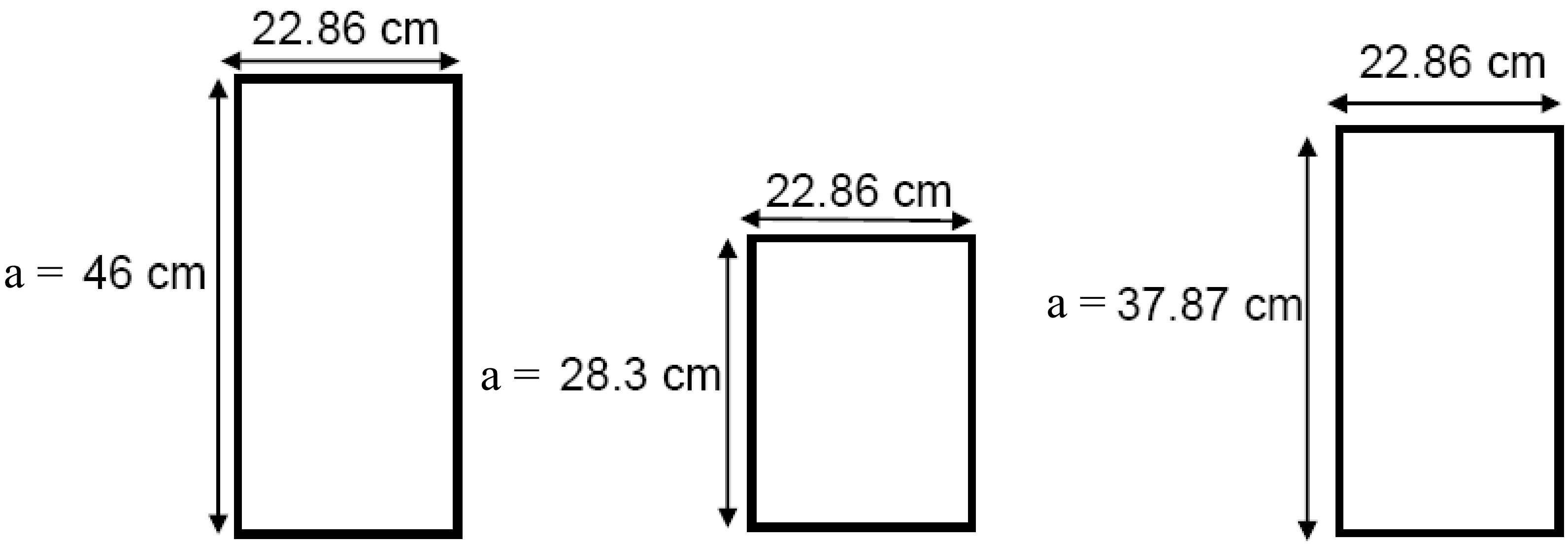
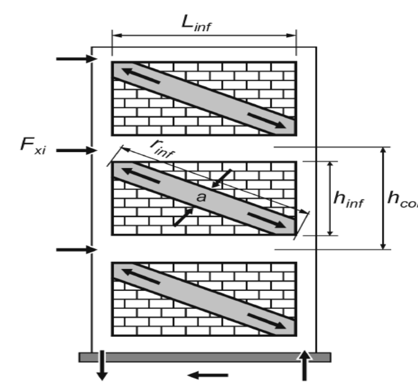


Table 7-1 of FEMA-356

FEMA-356 Single Strut Compression Model replace the Infill walls



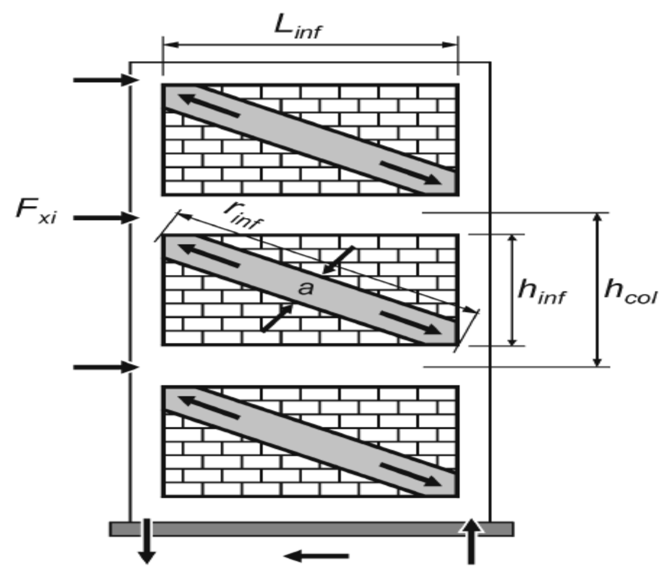


As the Masonry is unreinforced so the infill model has no tendency to take Tension Force only has the ability to take Compression force.

Table 7-1 of FEMA-356

•According to FEMA-356 a Nonlinear Hinge (Force Deformation Curve/Back Bone Curve) should be define to Capture the Nonlinear Behavior of the component of RC structure assigned for strut Model as shown in figure

•The Parameter used in calculation for strut properties to Replace the infill walls are shown in figure



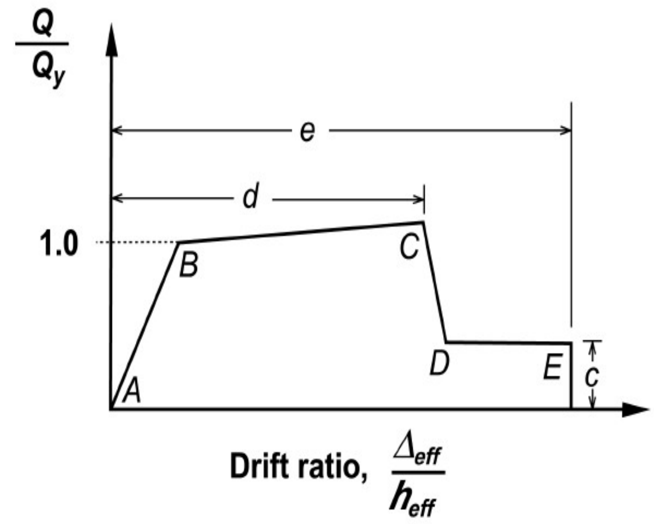
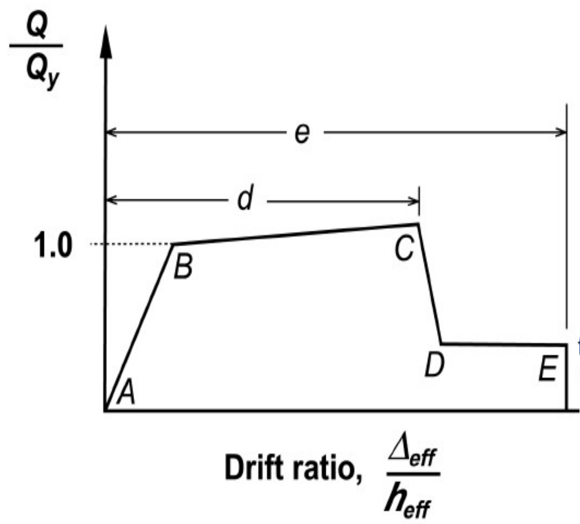


Table 7-9 of FEMA-356

Modeling (Component Level)



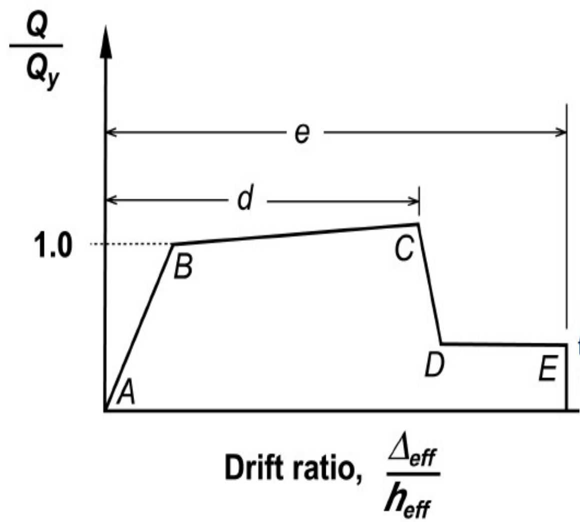
According to FEMA-356 Table 7-9 Different Limit states for the nonlinear hinge should be define

Table 7-9 Nonlinear Static Procedure—Simplified Force-Deflection Relations for Masonry Infill Panels

$\beta = \frac{V_{fre}}{V_{ine}}$	$\frac{L_{inf}}{h_{inf}}$	c	d %	e %	Acceptance Criteria LS %	Acceptance Criteria CP %
$\beta < 0.7$	0.5	n.a.	0.5	n.a.	0.4	n.a.
	1.0	n.a.	0.4	n.a.	0.3	n.a.
	2.0	n.a.	0.3	n.a.	0.2	n.a.
$0.7 \le \beta < 1.3$	0.5	n.a.	1.0	n.a.	0.8	n.a.
	1.0	n.a.	0.8	n.a.	0.6	n.a.
	2.0	n.a.	0.6	n.a.	0.4	n.a.
$\beta \ge 1.3$	0.5	n.a.	1.5	n.a.	1.1	n.a.
	1.0	n.a.	1.2	n.a.	0.9	n.a.
	2.0	n.a.	0.9	n.a.	0.7	n.a.

Note: Interpolation shall be used between table values.

Table 7-9 of FEMA-356



The Back Bone Curve for the infill model should be define in the computer Model as shown in the Figure

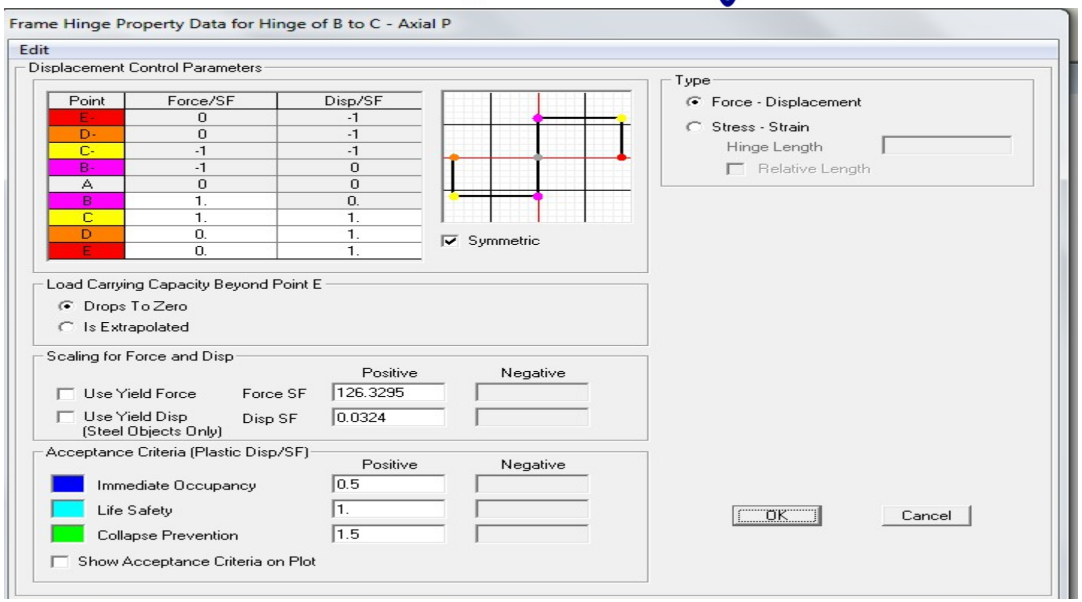
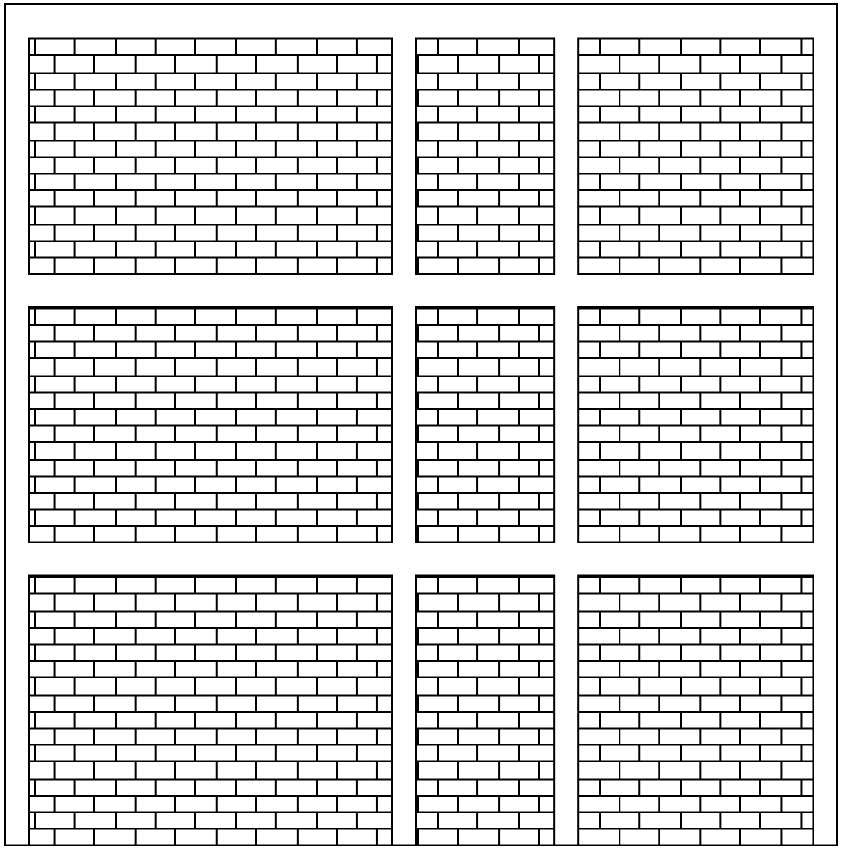


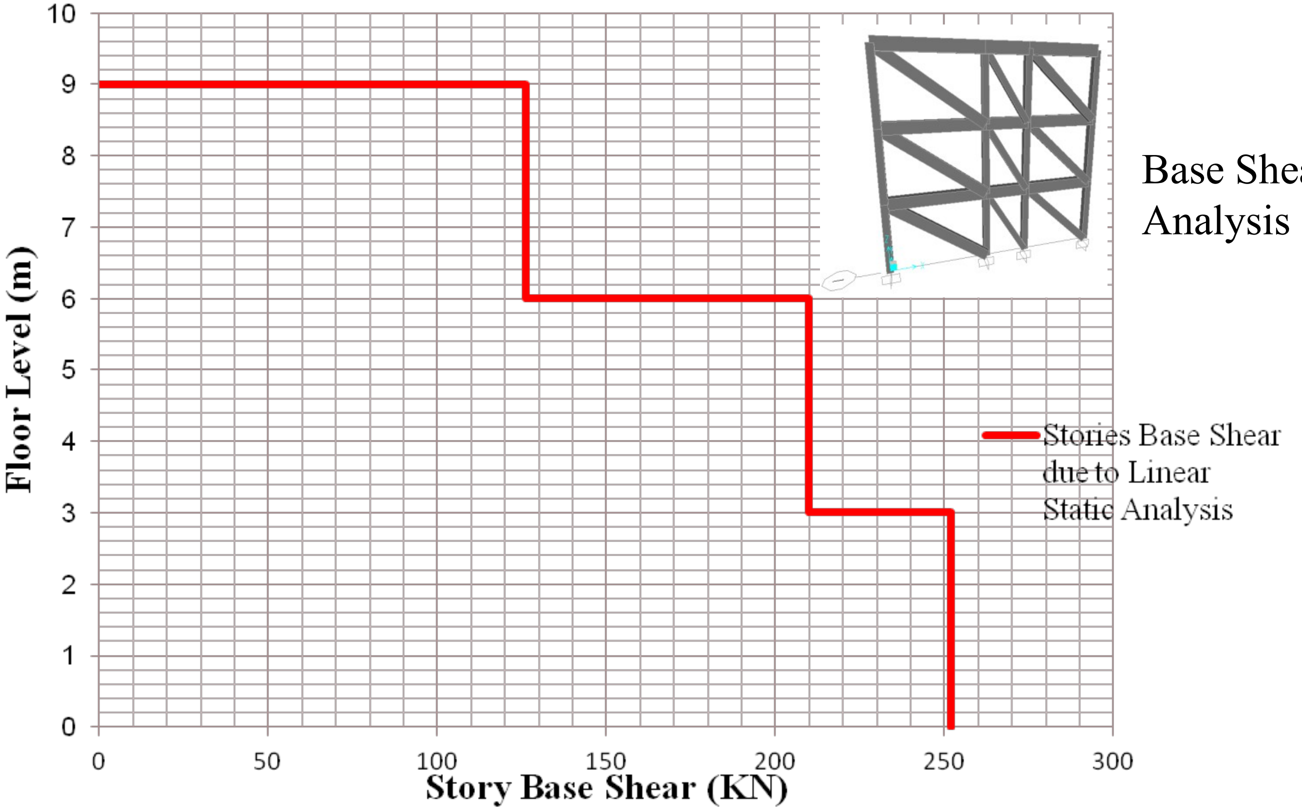
Table 7-9 of FEMA-356

Linear Static Analysis

FULL INFILLL 2-D FRAME



Linear Static Analysis (System Response)



Base Shear By Linear Static Analysis is 250 KN

Linear Static Analysis

(System Response)

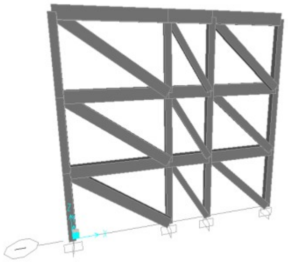
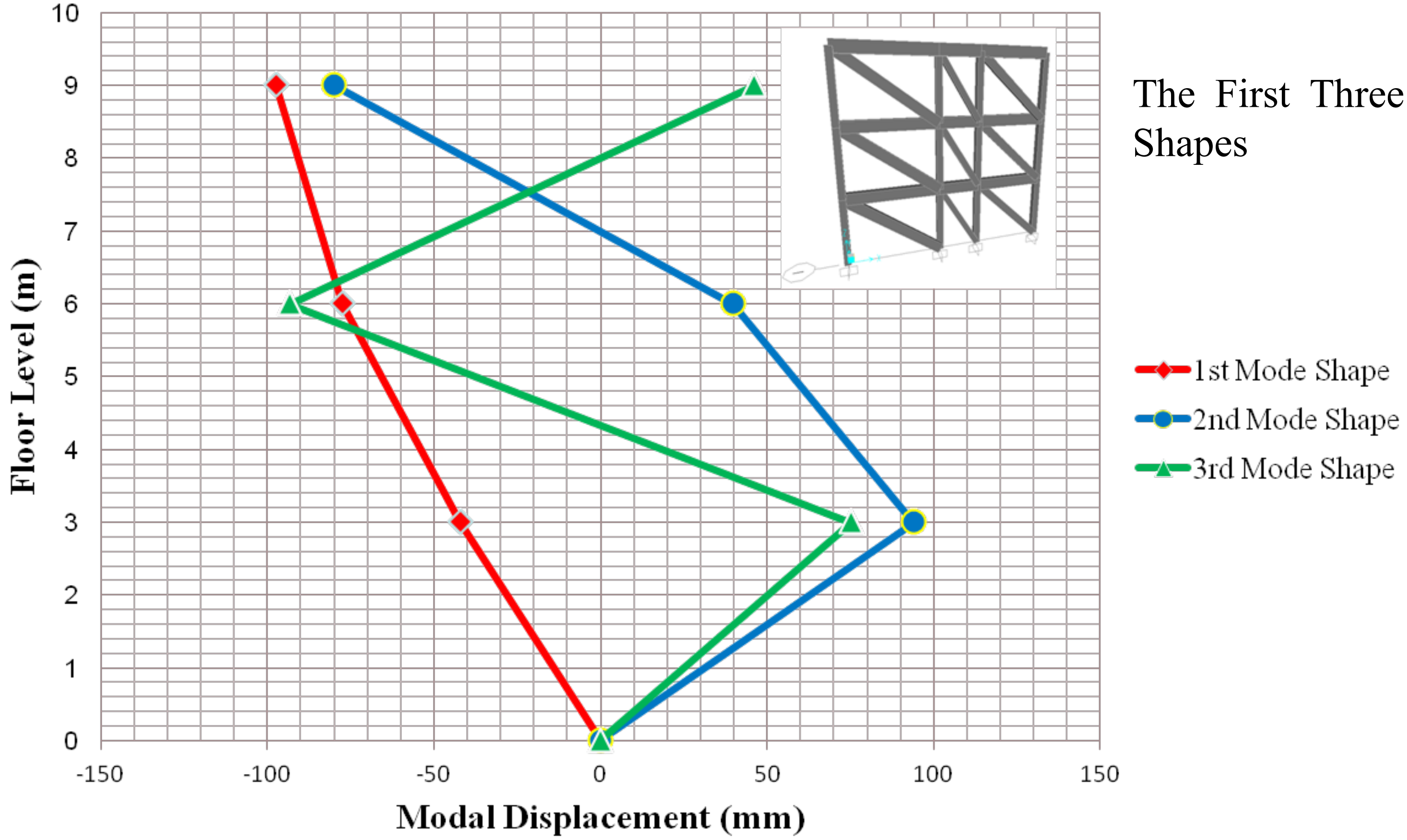


TABLE: Modal Participating Mass Ratios

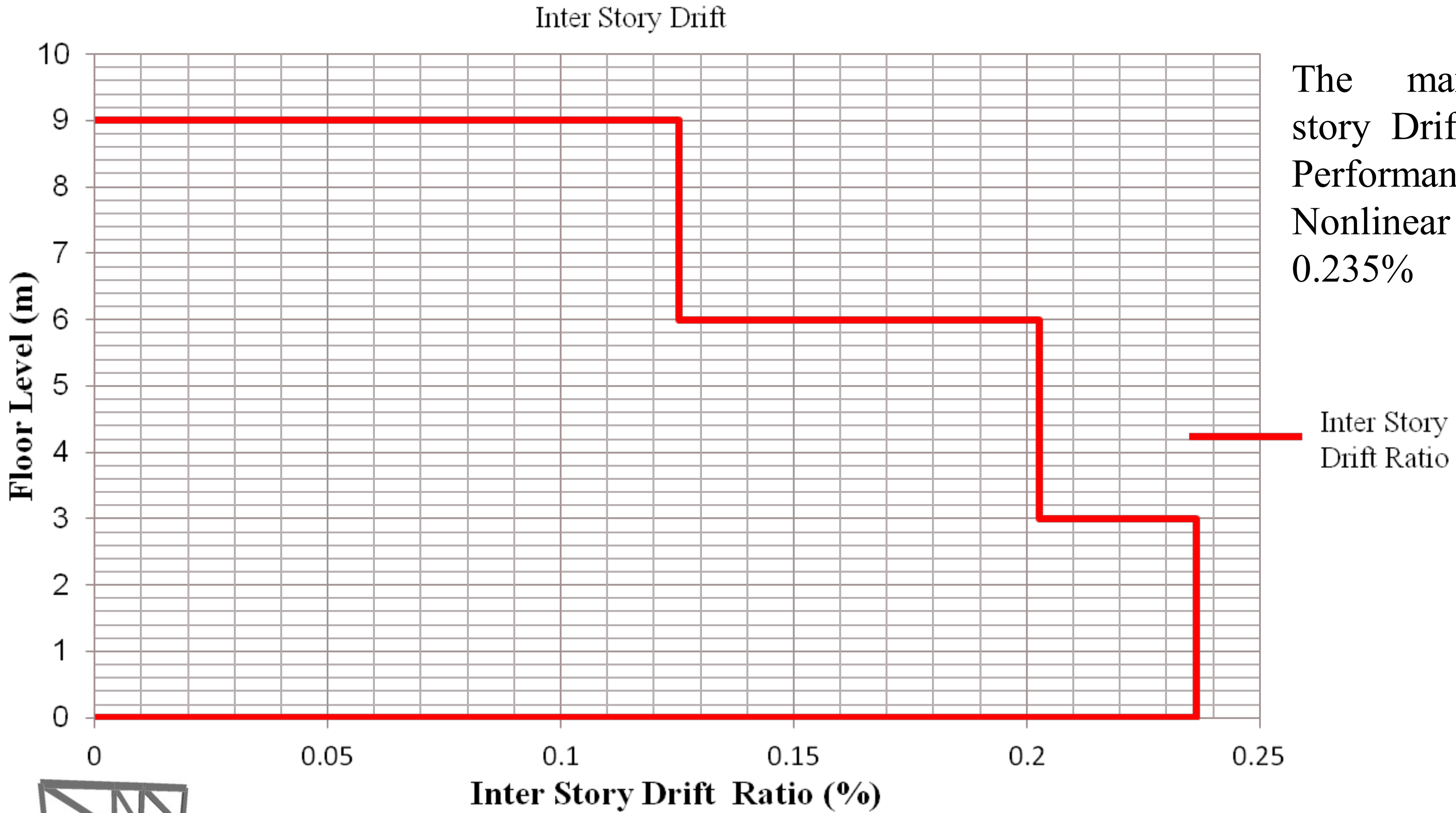
OutputCase	**StepType**	**StepNum**	**Period**	**UX**	**UY**	**UZ**	**SumUX**	**SumUY**	**SumUZ**	**RX**	**RY**	**RZ**	**SumRX**	**SumRY**	**SumRZ**
Text	Text	Unitless	Sec	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
MODAL	Mode	1	0.38699	0.91	0	0.0003832	**0.91**	0	0.000383	0	0.53	0	0	0.53	0
MODAL	Mode	2	0.13805	0.08145	0	0.0000041	**0.99**	0	0.000387	0	0.007793	0	0	0.53	0
MODAL	Mode	3	0.09456	0.01178	0	0.0000339	**1**	0	0.000421	0	0.0006998	0	0	0.53	0

Linear Static Analysis (System Response)

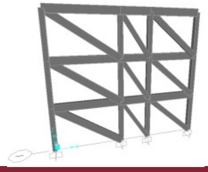


The First Three Effective Modes Shapes

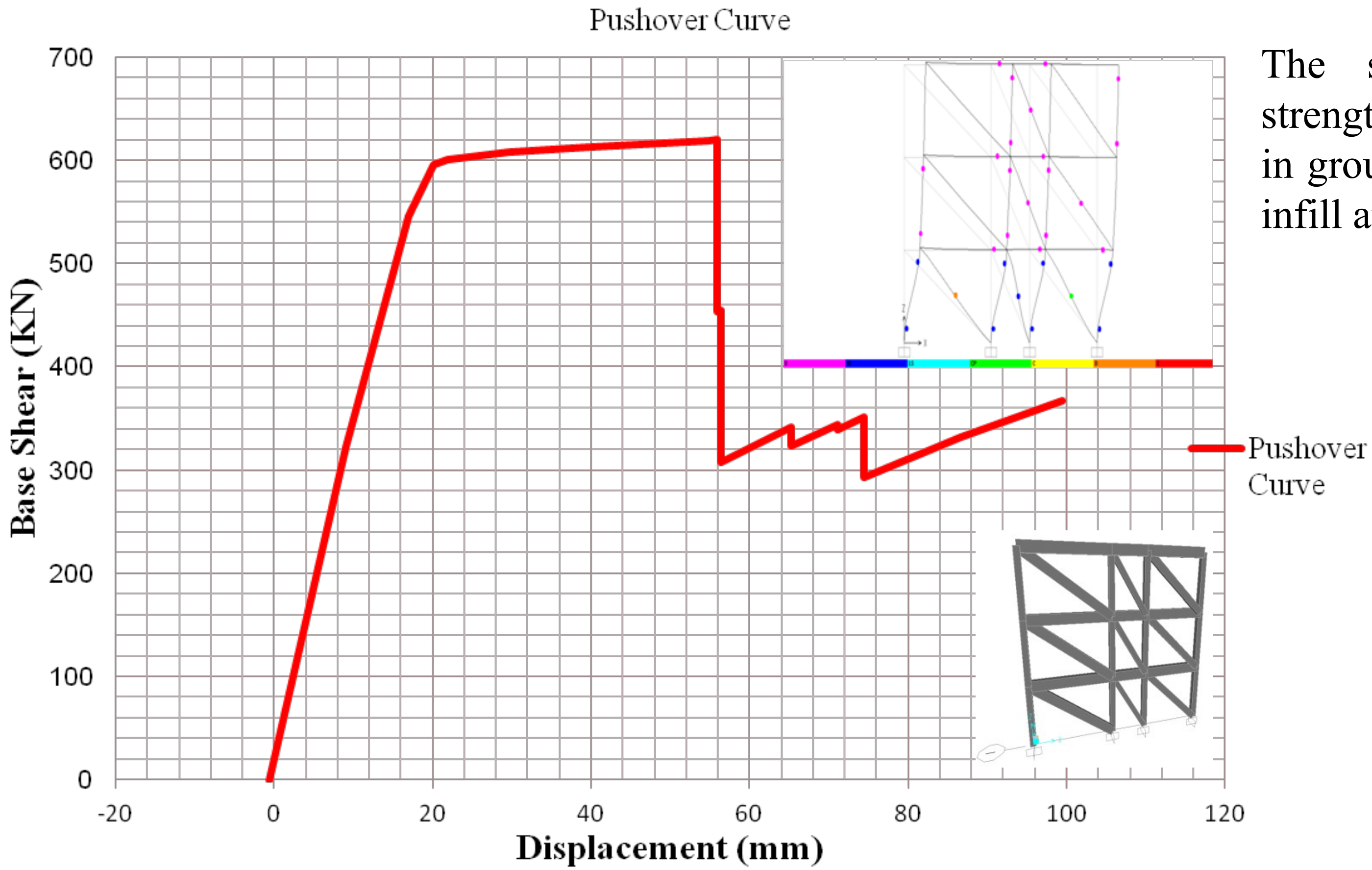
Nonlinear Pushover Analysis (System Response)



The maximum inter story Drift Ratio at the Performance level by Nonlinear Analysis is 0.235%

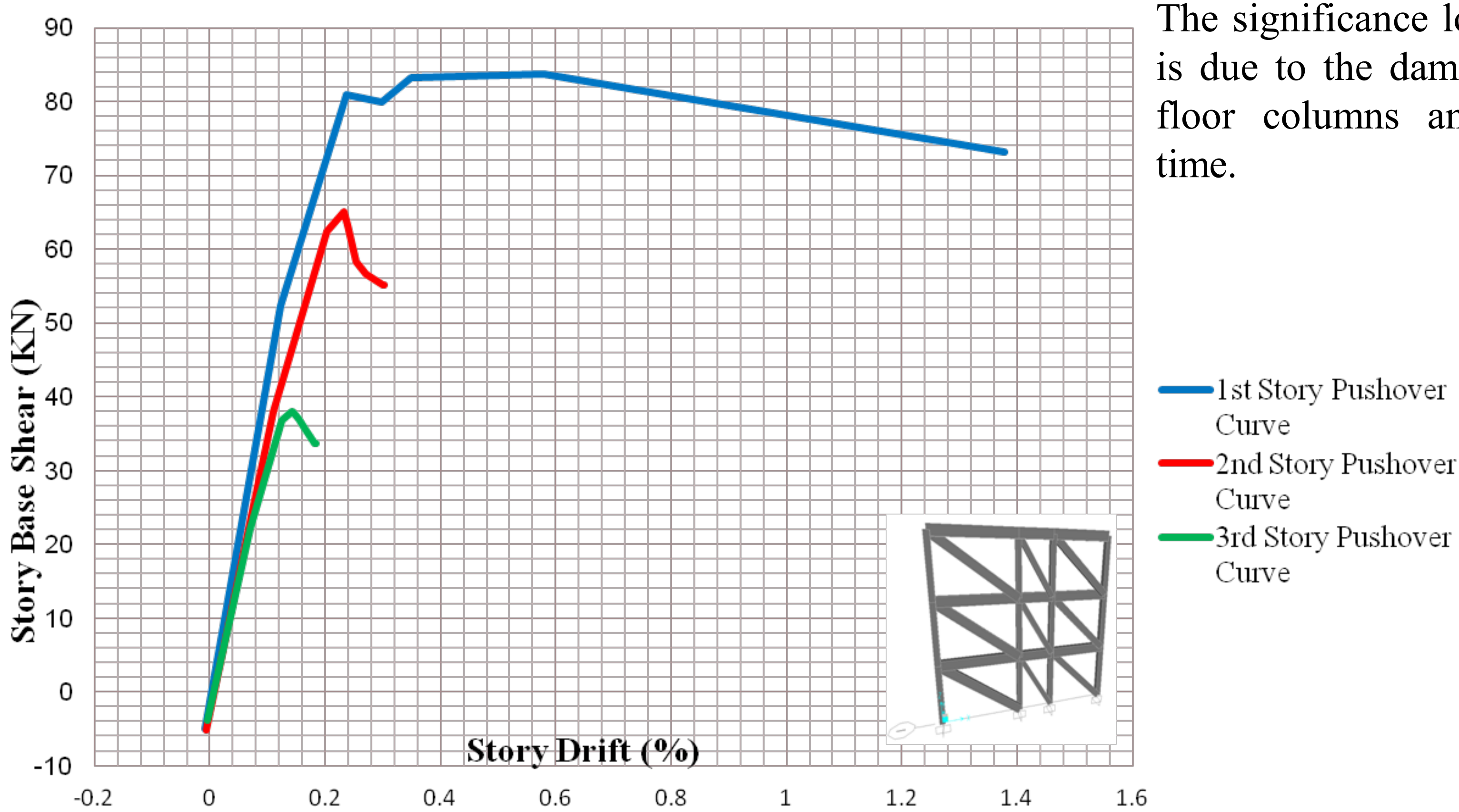


Nonlinear Pushover Analysis (System Response)



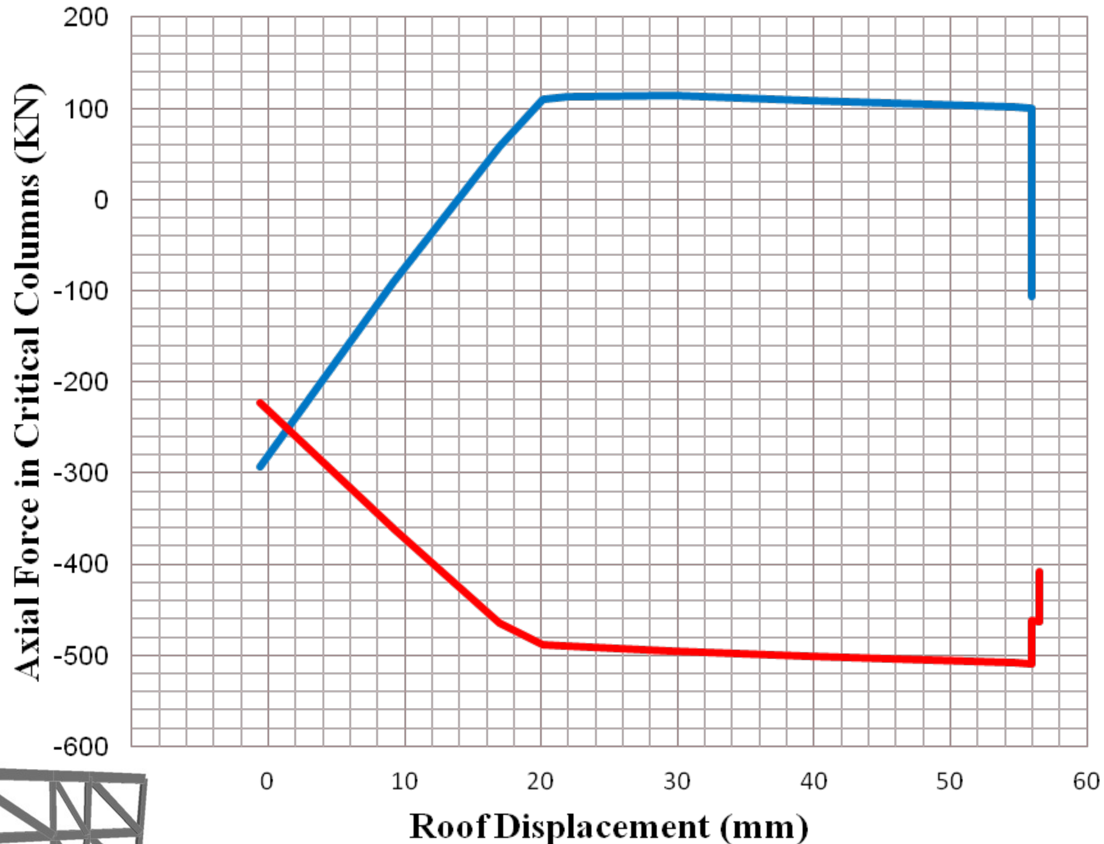
The significance loss in strength is due to the damage in ground floor columns and infill at a time.

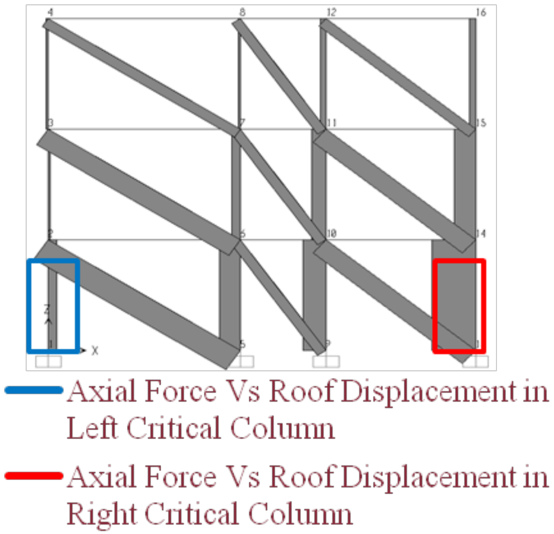
Nonlinear Pushover Analysis (System Response)



The significance loss in strength is due to the damage in ground floor columns and infill at a time.

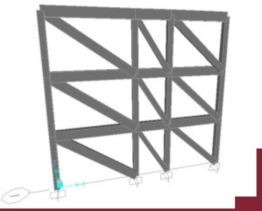
Nonlinear Pushover Analysis (Components Response)



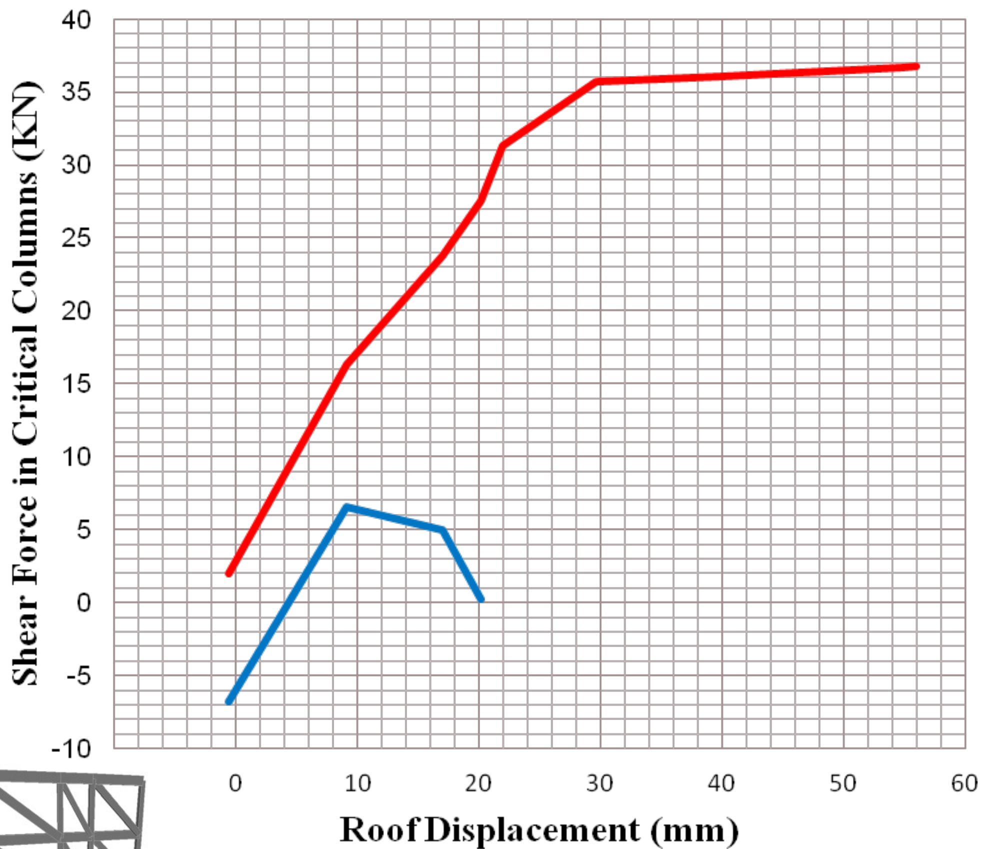


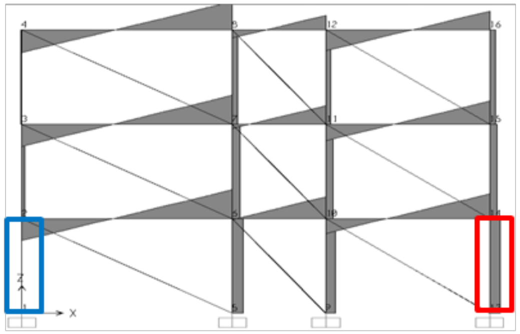
The maximum Axial force in Left column is 100 KN having displacement 56 mm

The maximum Axial force in the Right column is -508 KN(compression) having Roof displacement 56 mm



Nonlinear Pushover Analysis (Components Response)



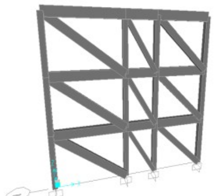


Shear Force Vs Roof Displacement in Left Critical Column

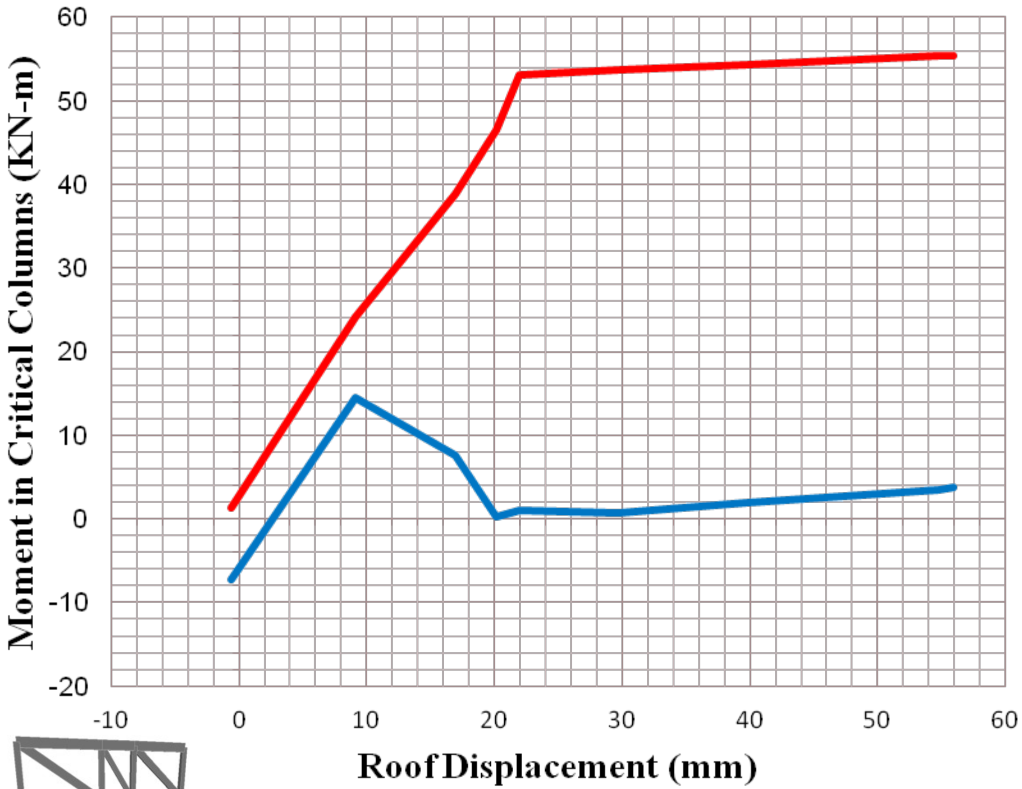
Shear Force Vs Roof Displacement in Right Critical Column

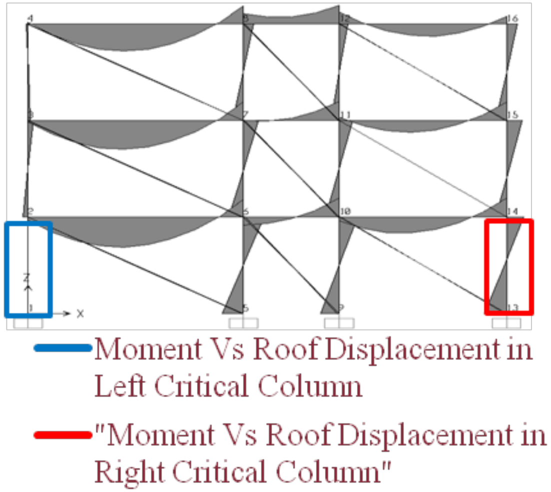
The maximum Shear Force in Left Column is 6.5 KN

The maximum Shear Force in Right Column is 37 KN



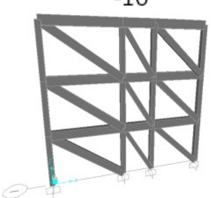
Nonlinear Pushover Analysis (Components Response)



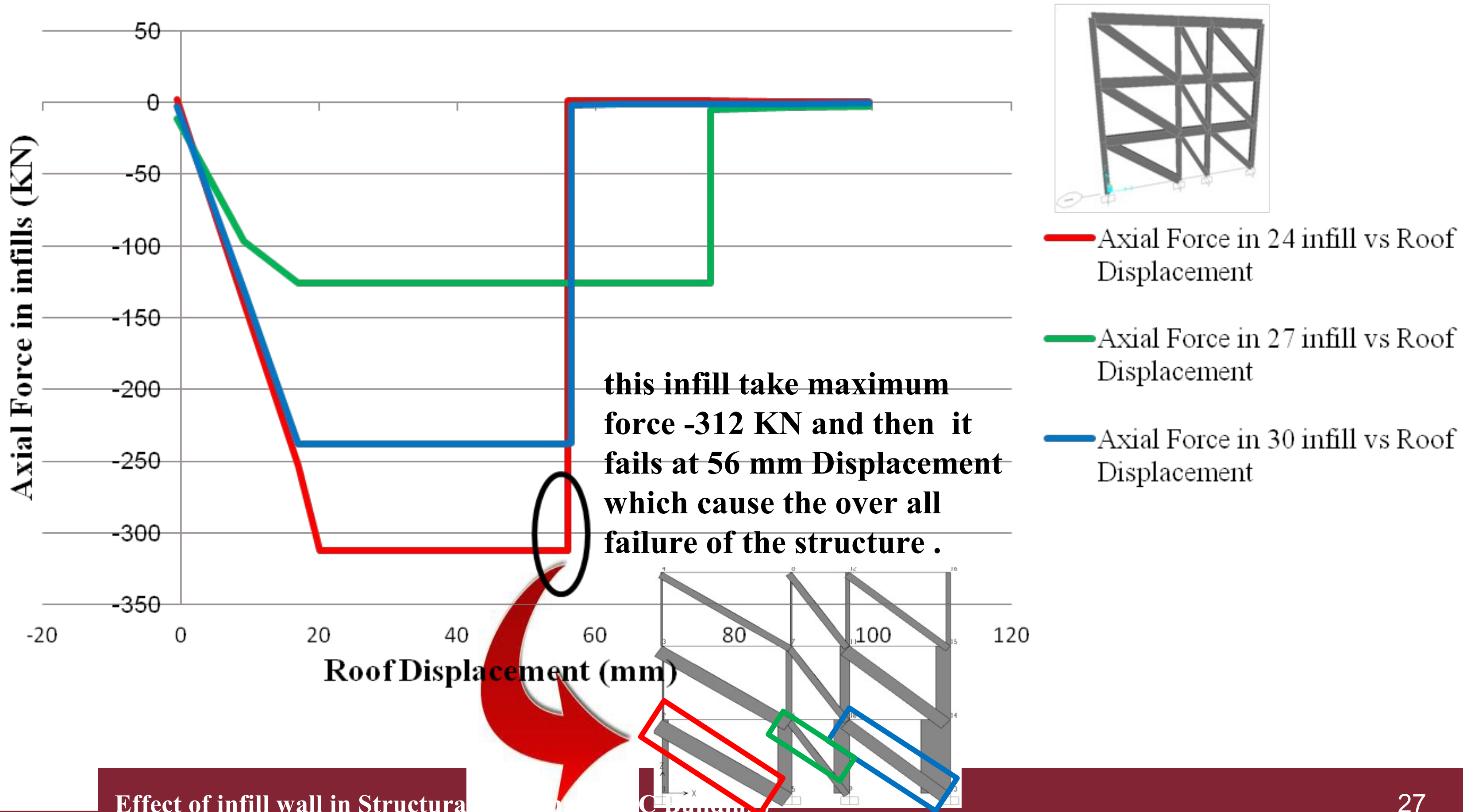


The Maximum Moment at the Left side Column is 14.5 KN-m

The maximum Moment at the Right Hand side column is 55.5 KN-m

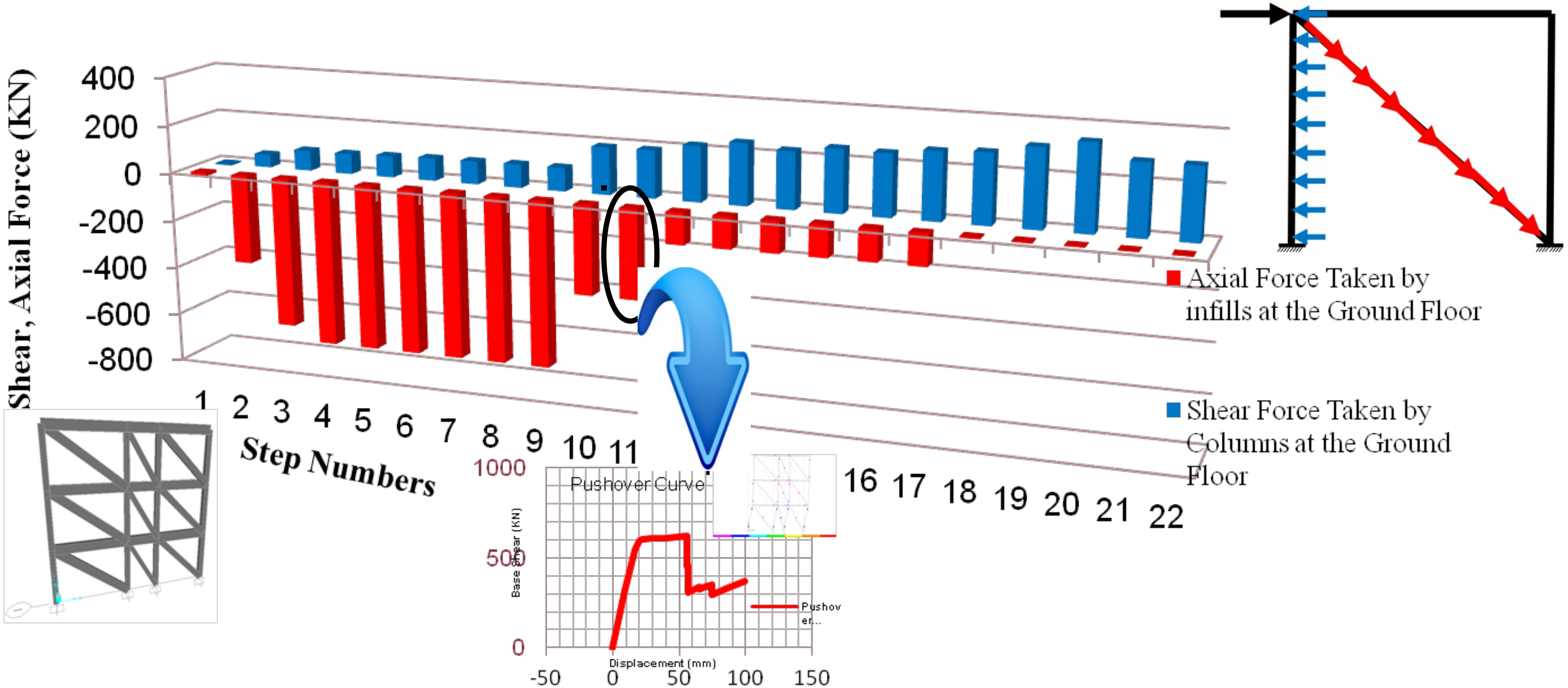


Nonlinear Pushover Analysis (Components Response)



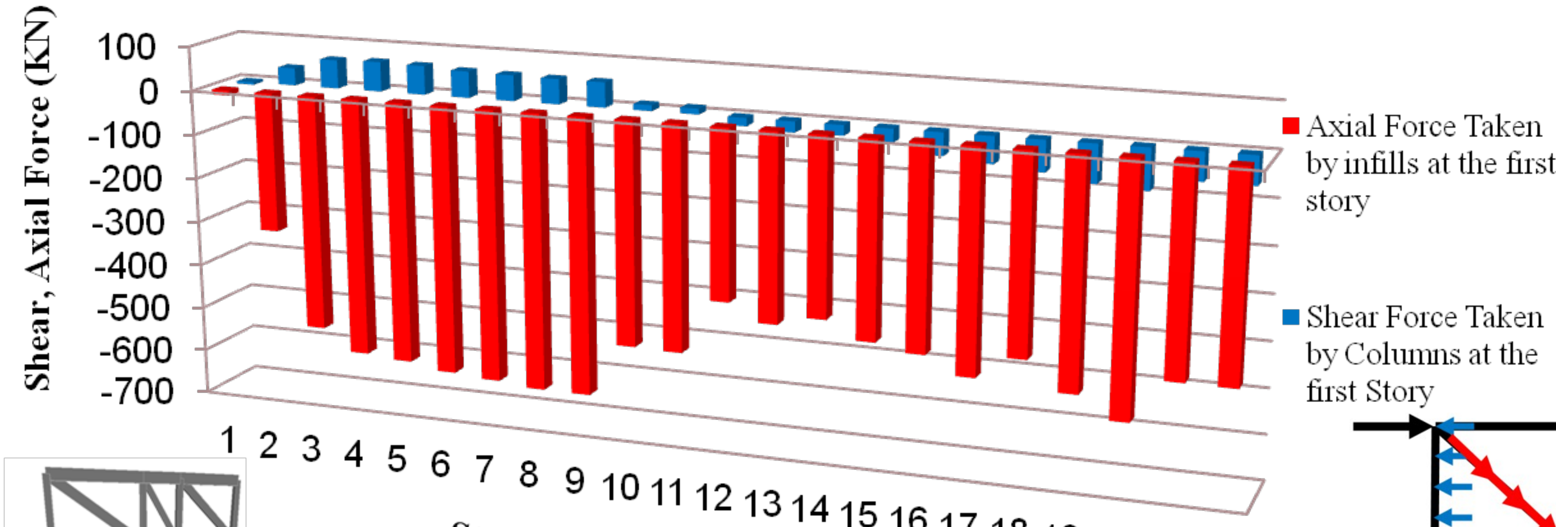
Nonlinear Pushover Analysis (Components Response)

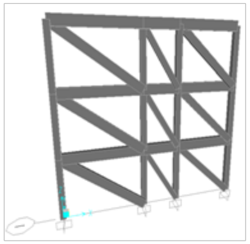
Contribution of Infills and columns in taking shear Force at The Ground story
The Circle one is the force taken by infills at 11th step and then it fails at next step, because of this failure the Pushover Curve Drops Sudden as shown in figure

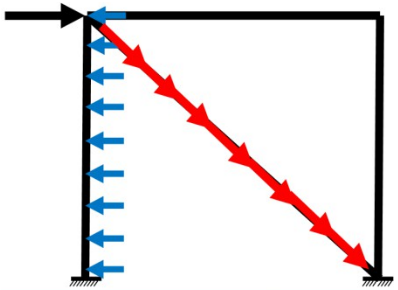


Nonlinear Pushover Analysis (Components Response)

Contribution of Infill and columns in taking shear Force at The 1st story

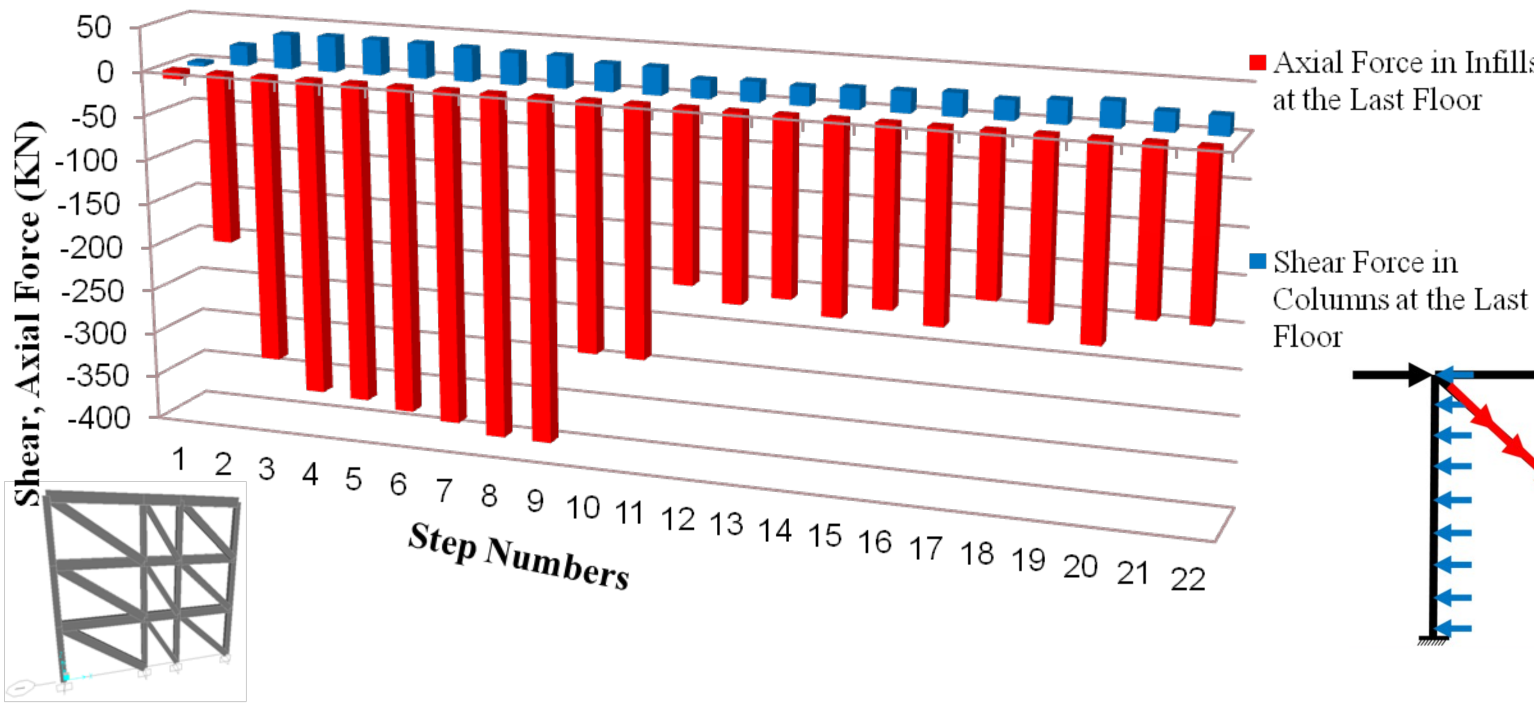


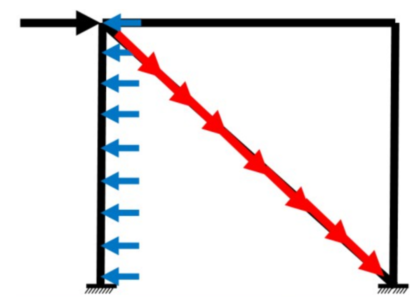




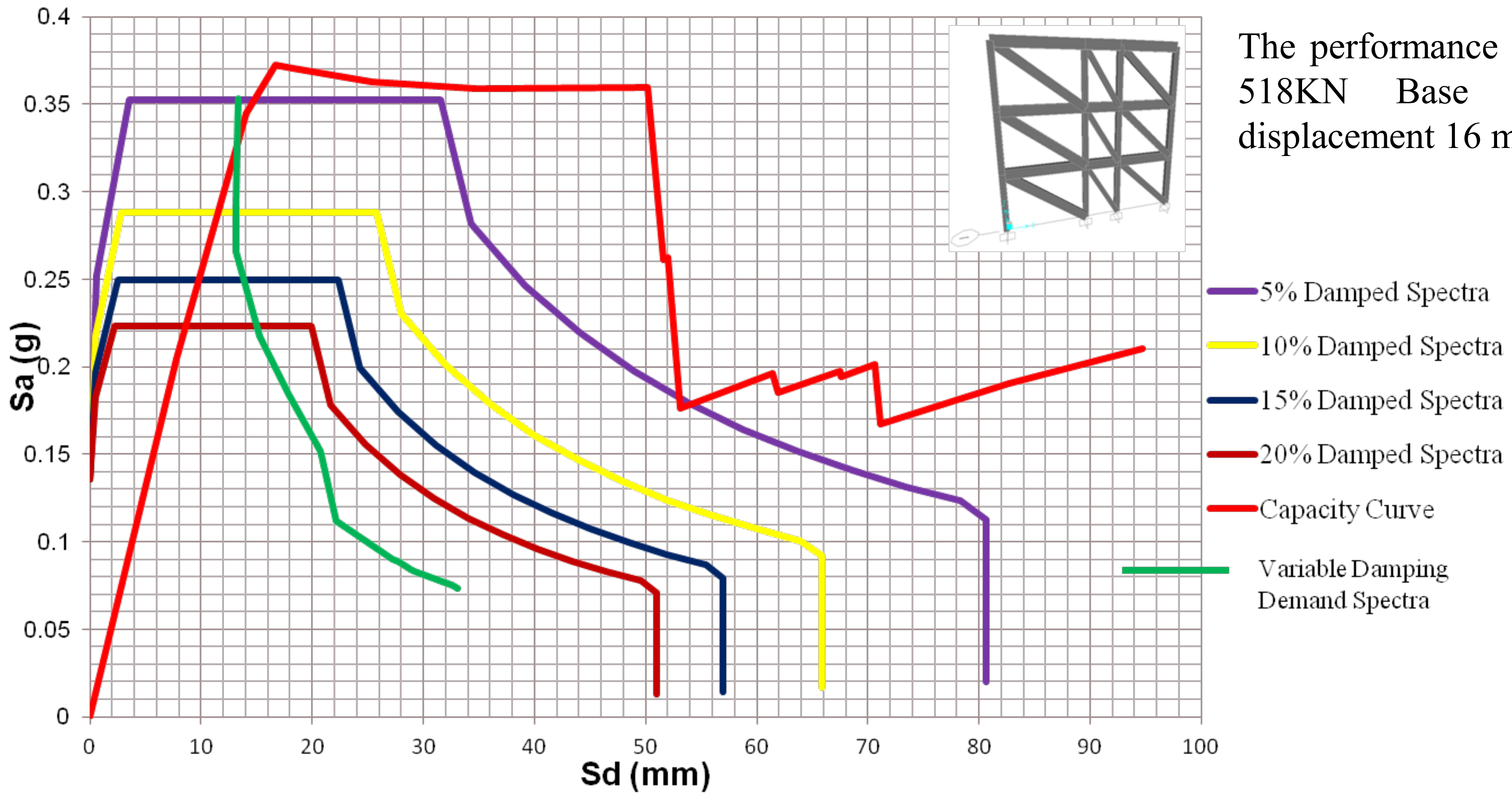
Nonlinear Pushover Analysis (Components Response)

Contribution of Infill and columns in taking shear Force at The Last story





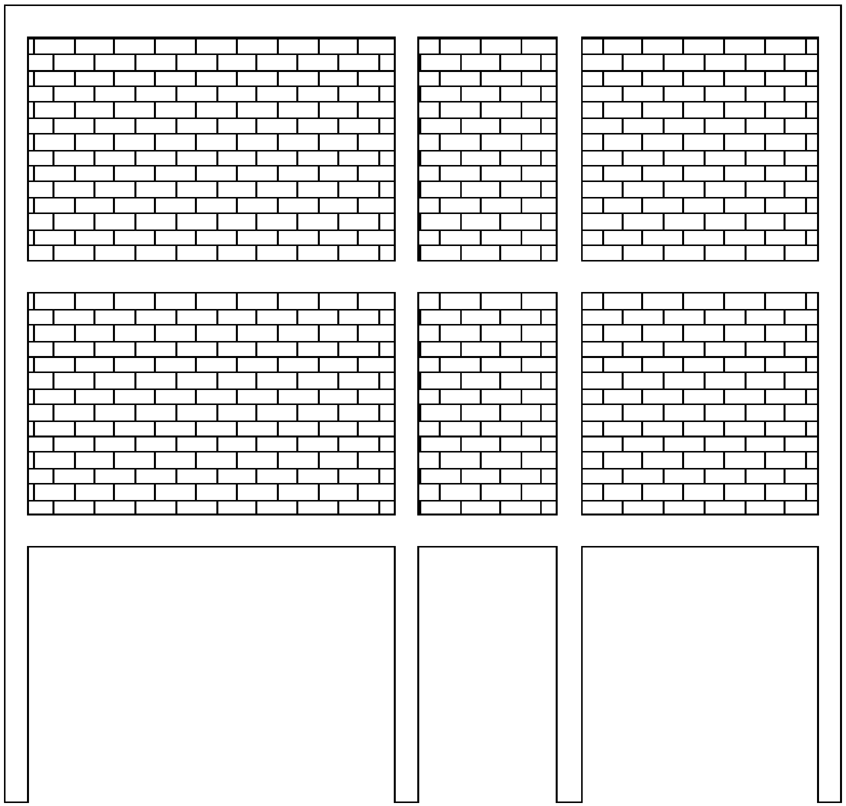
Use of Pushover Curve (ATC-40) (System Response)



The performance point is at 518KN Base shear at displacement 16 mm

Linear Static Analysis

SOFT STORY INFILLL 2-D FRAME



Linear Static Analysis (System Response)

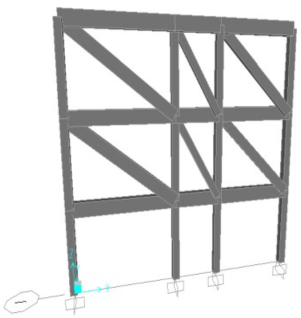
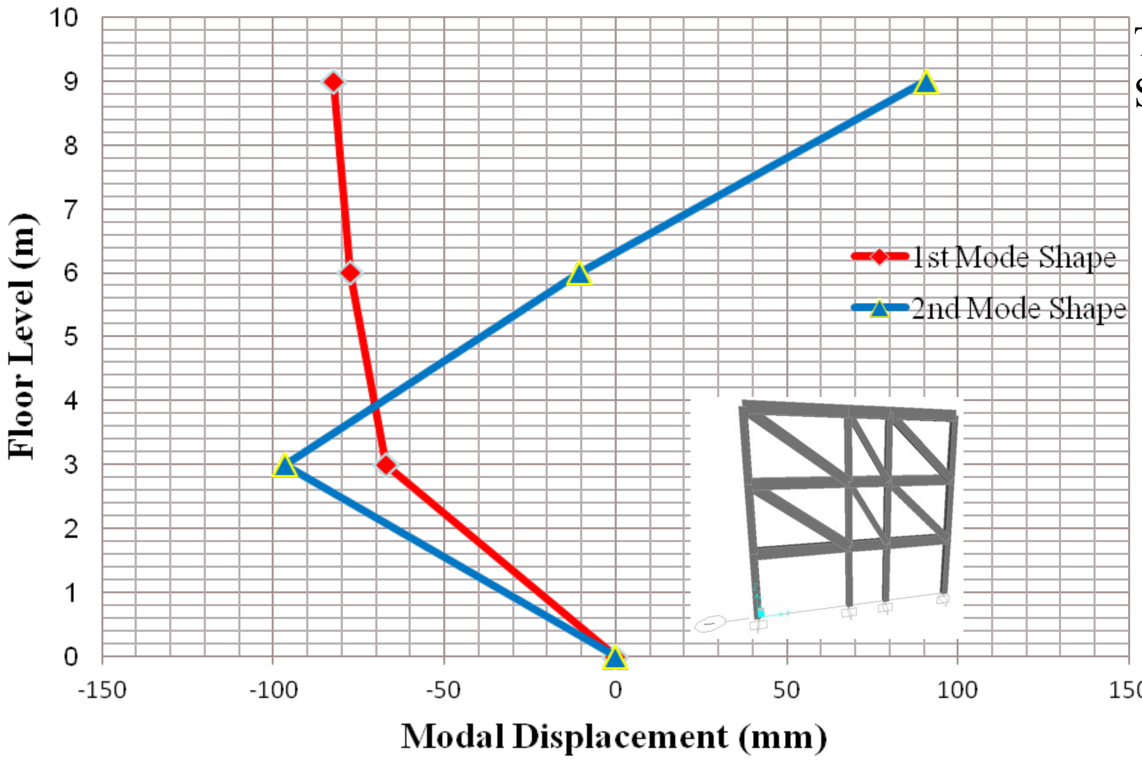


TABLE: Modal Participating Mass Ratios

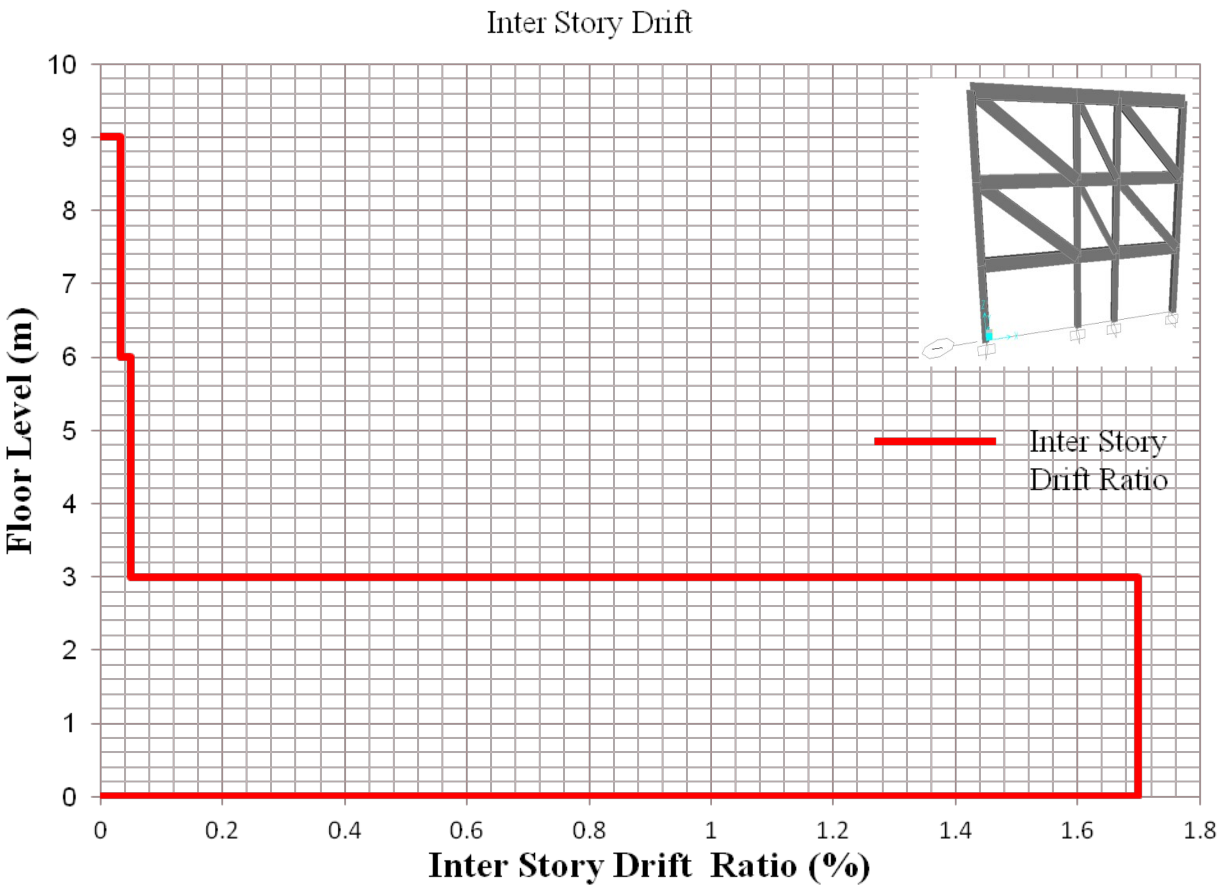
OutputCase	StepType	StepNum	Period	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX	SumRY	SumRZ
Text	Text	Unitless	Sec	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
MODAL	Mode	1	0.71186	0.99	0	0.000007012	0.99	0	0.0000070	0	0.49	0	0	0.49	0
MODAL	Mode	2	0.16193	0.006648	0	0.001024	1	0	0.001031	0	0.05017	0	0	0.54	0

Linear Static Analysis (System Response)



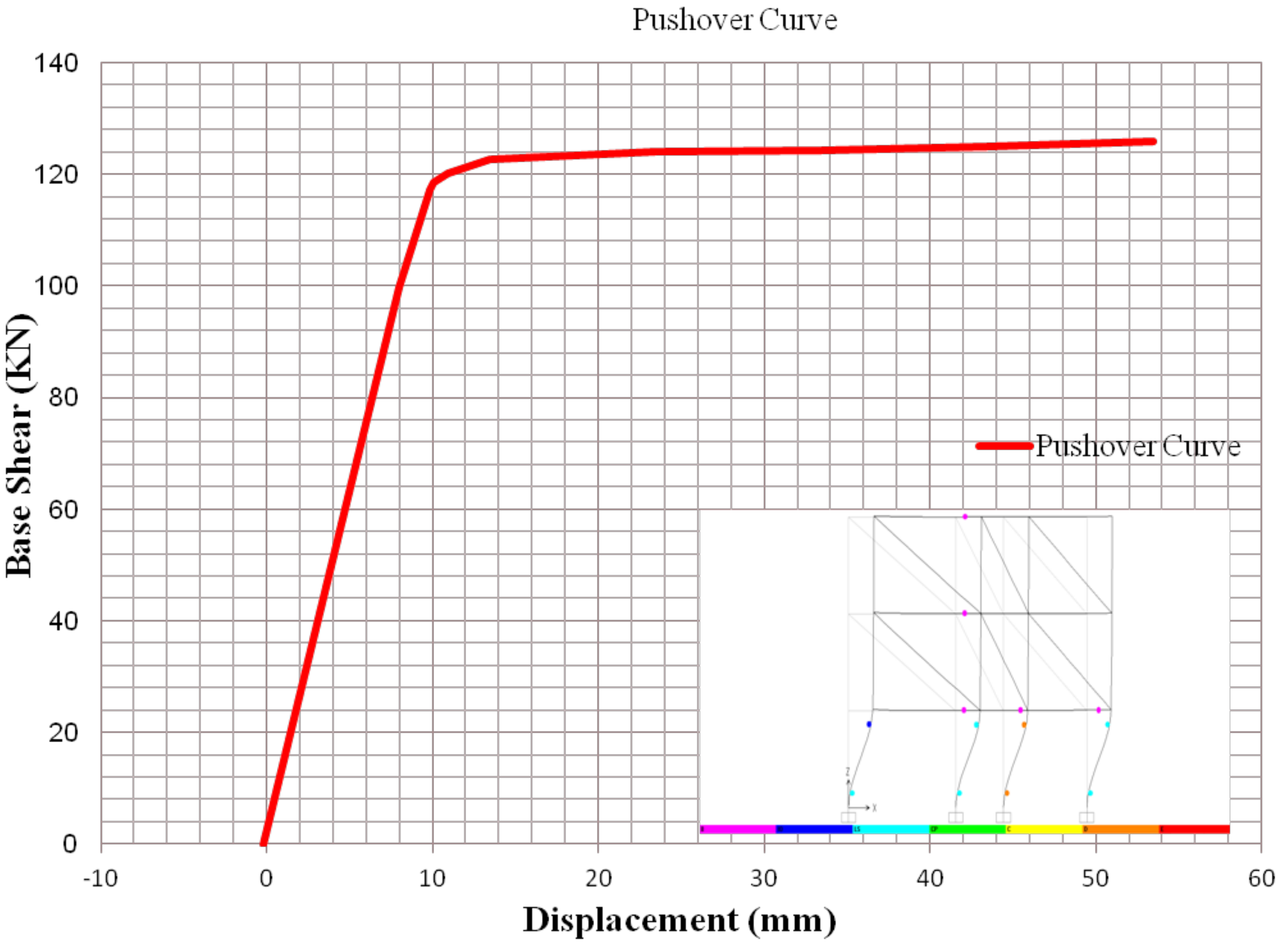
The First Two Effective Modes Shapes

Nonlinear Pushover Analysis (System Response)

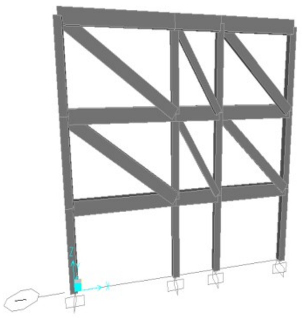


The maximum inter story Drift Ratio at the Performance level by Nonlinear Analysis is 1.7%

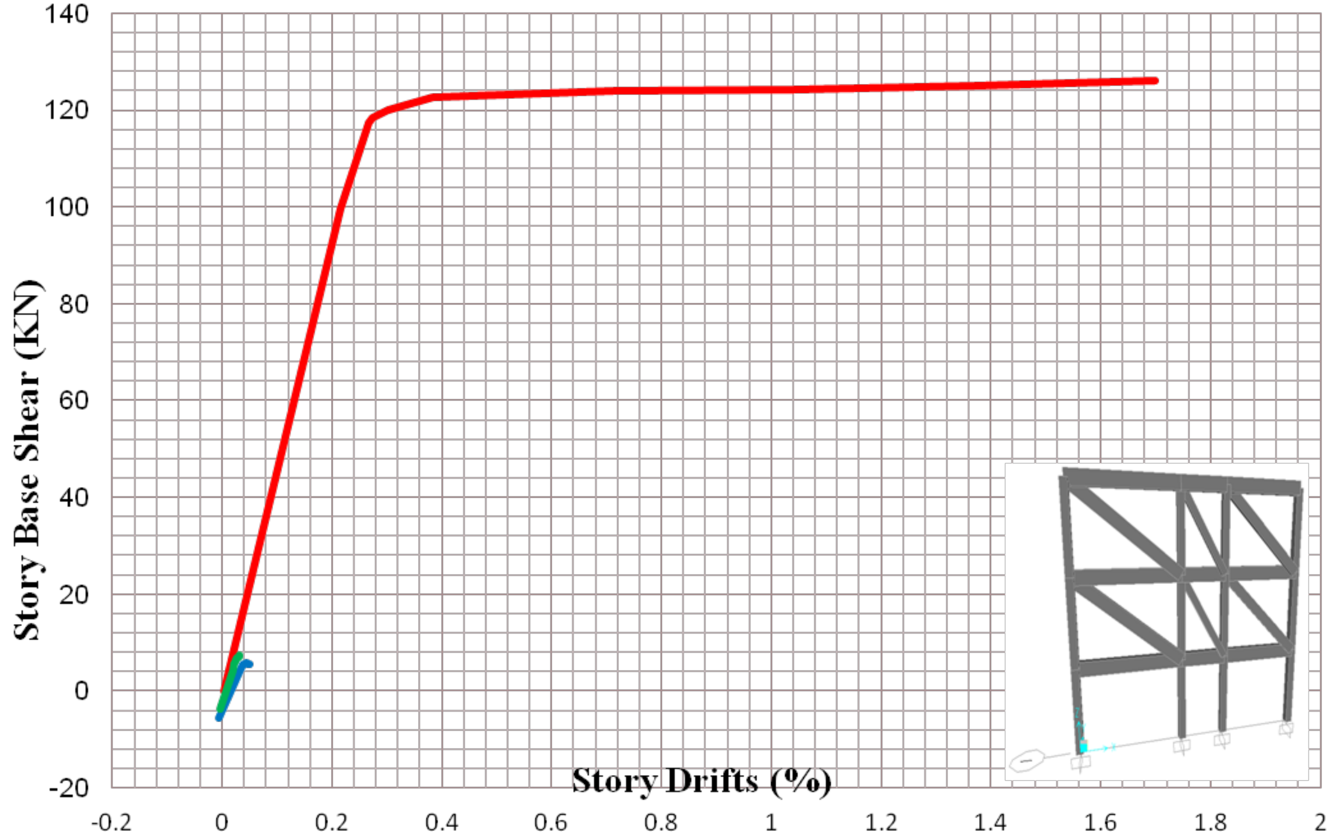
Nonlinear Pushover Analysis (System Response)



The structure collapse at 9th step of push load case at Base Shear 126 KN having displacement 53 mm because of the columns failure at the ground story level.

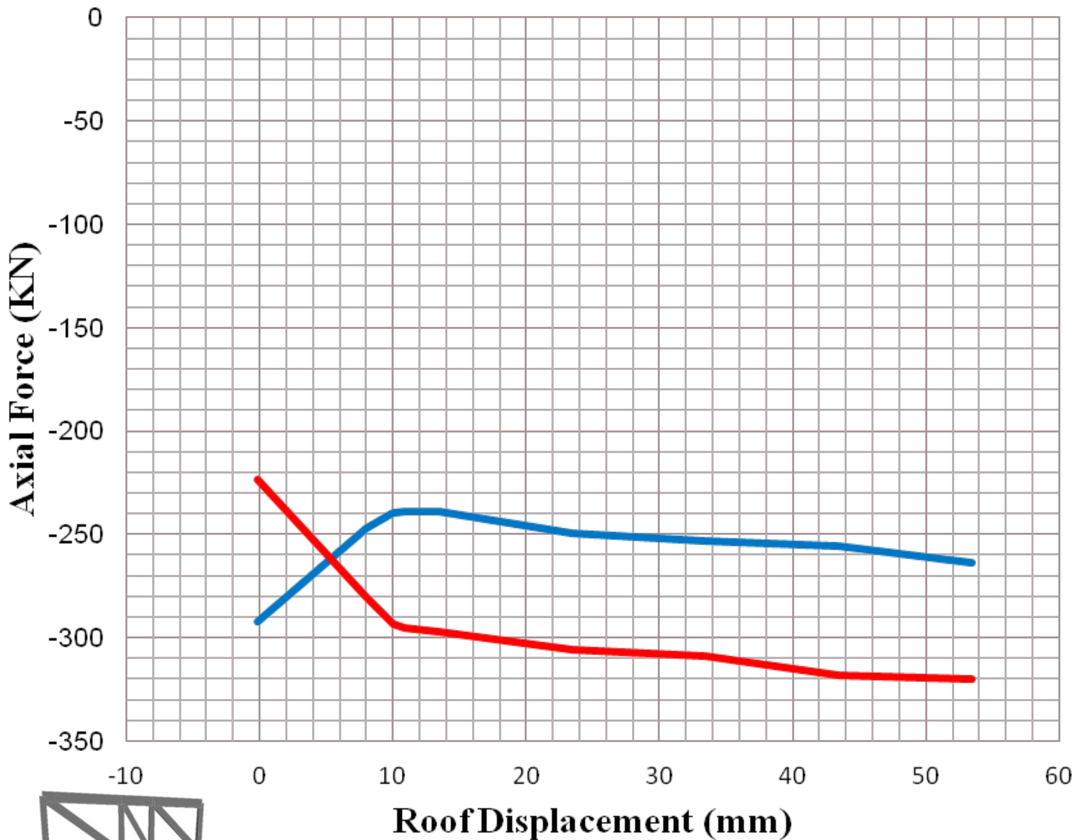


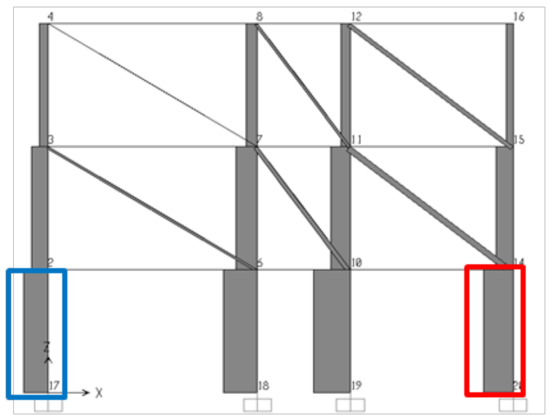
Nonlinear Pushover Analysis (System Response)



The structure collapse having Base Shear 126 KN and displacement 53 mm because of the columns failure at the (Soft Story) ground story level.

Nonlinear Pushover Analysis (Components Response)



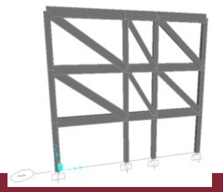


Axial Force in Left Column vs Roof Displacement

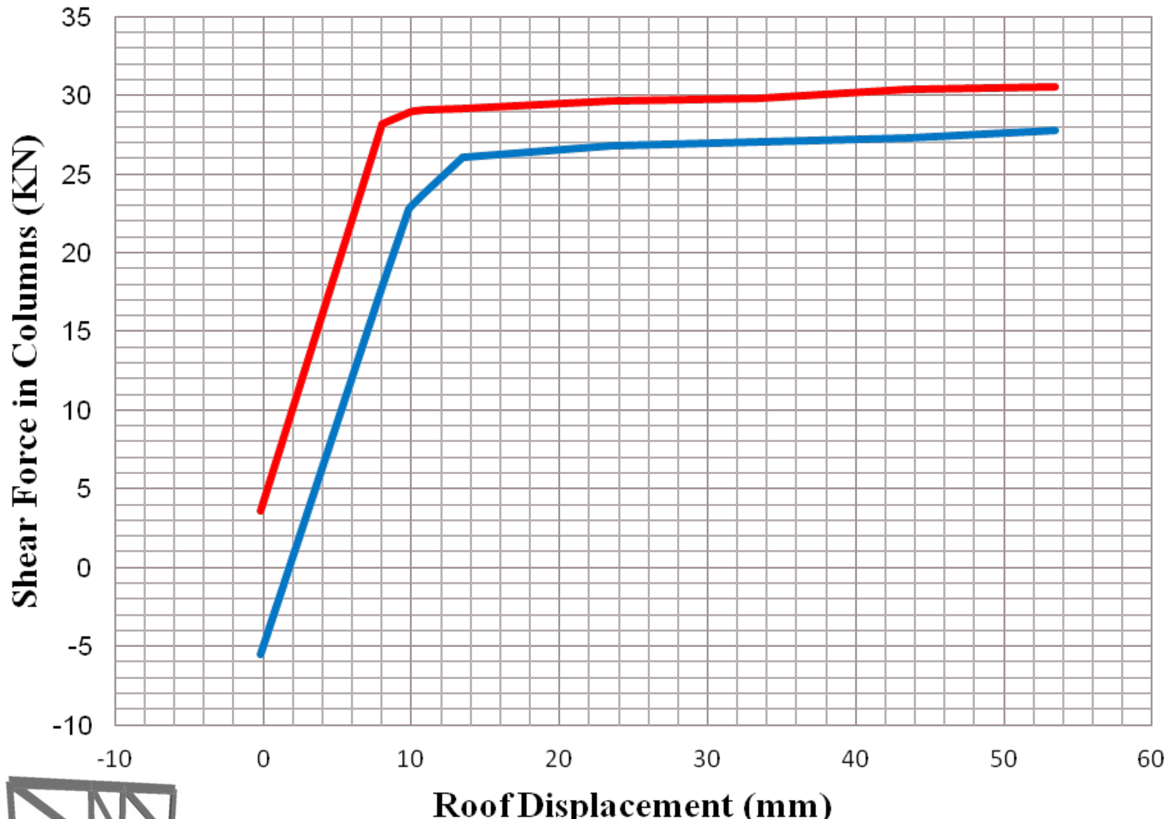
Axial Force in Right Column vs Roof Displacement

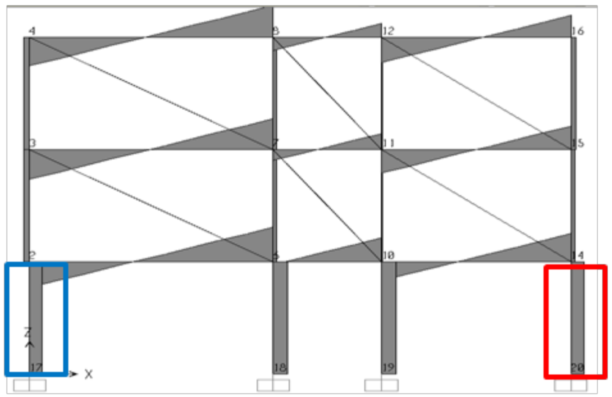
The maximum axial force in Left column is -264 KN

The Maximum Axial Force in Right column is -320 KN



Nonlinear Pushover Analysis (Components Response)



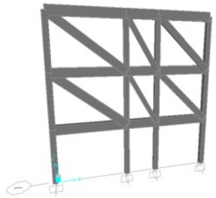


Shear Force in Left Column Vs Roof Displacement

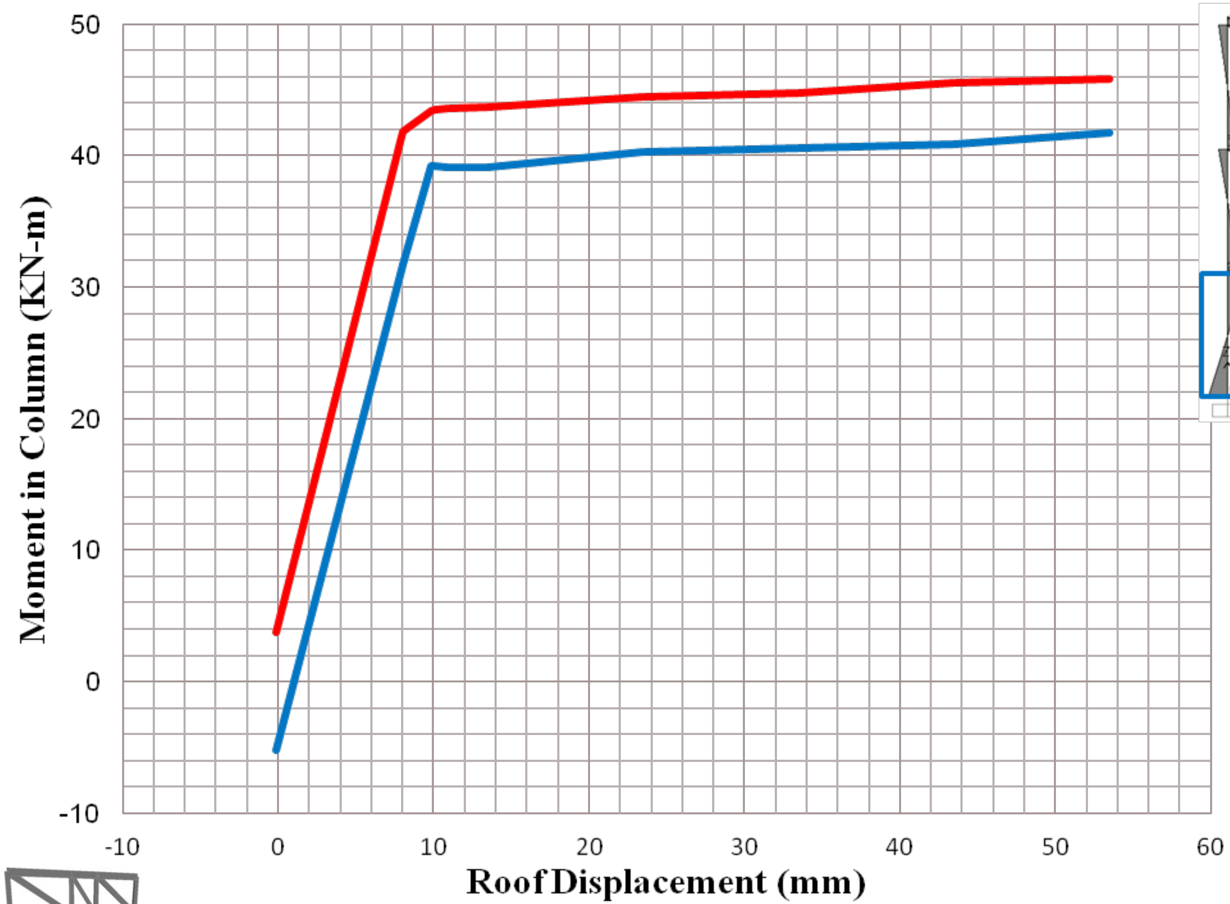
Shear Force in Right Column Vs Roof Displacement

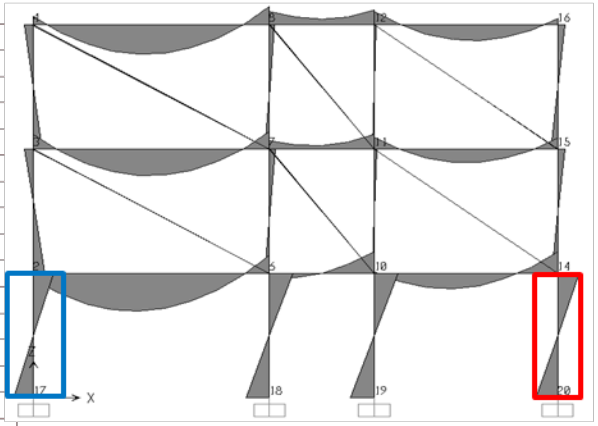
The maximum Shear Force in Left Column is 27.82 KN

The Maximum Shear Force in Right Column is 30.5 KN



Nonlinear Pushover Analysis (Components Response)



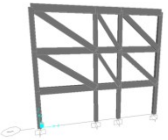


Moment in Left Column vs Roof Displacement

Moment in Right Column vs Roof Displacement

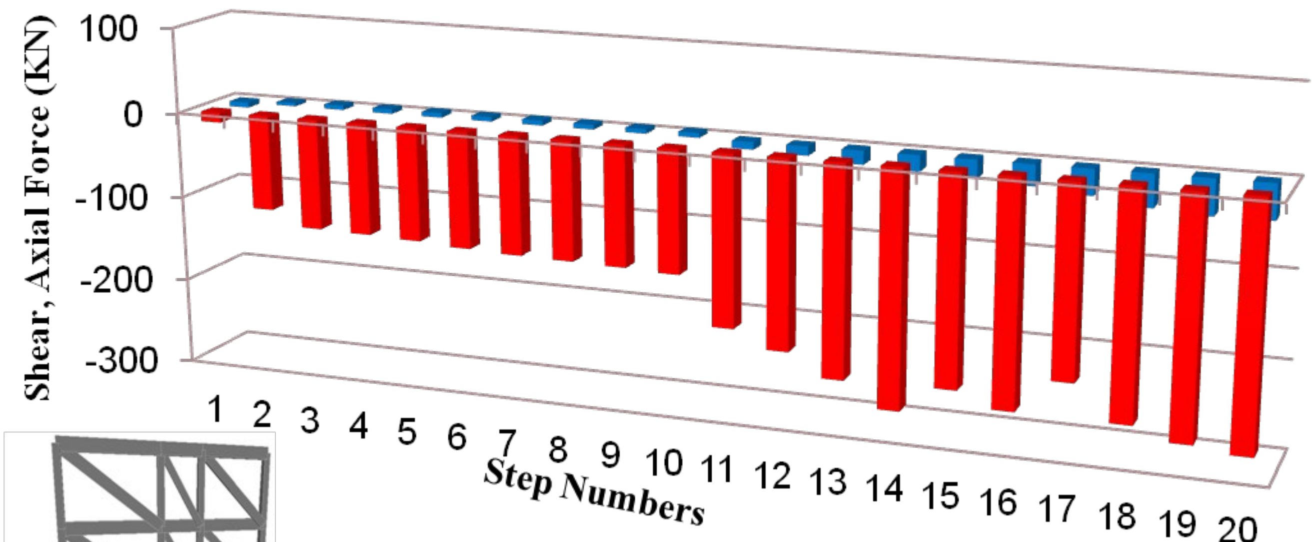
The maximum Moment in Left column is 42 KN-m

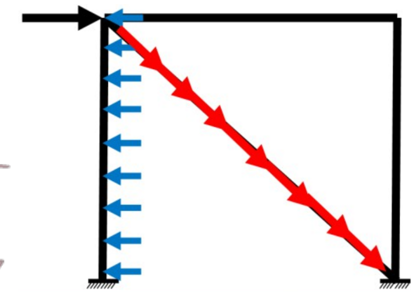
The Maximum Moment in Right column is 46 KN-m



Nonlinear Pushover Analysis (Components Response)

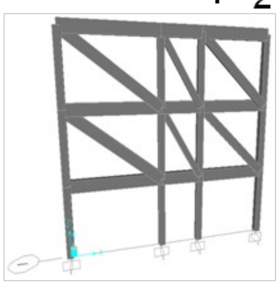
Contribution of Infill and columns in taking shear Force at The 1st story, the infill do not fail but the global failure of the frame is due to the soft story columns failure





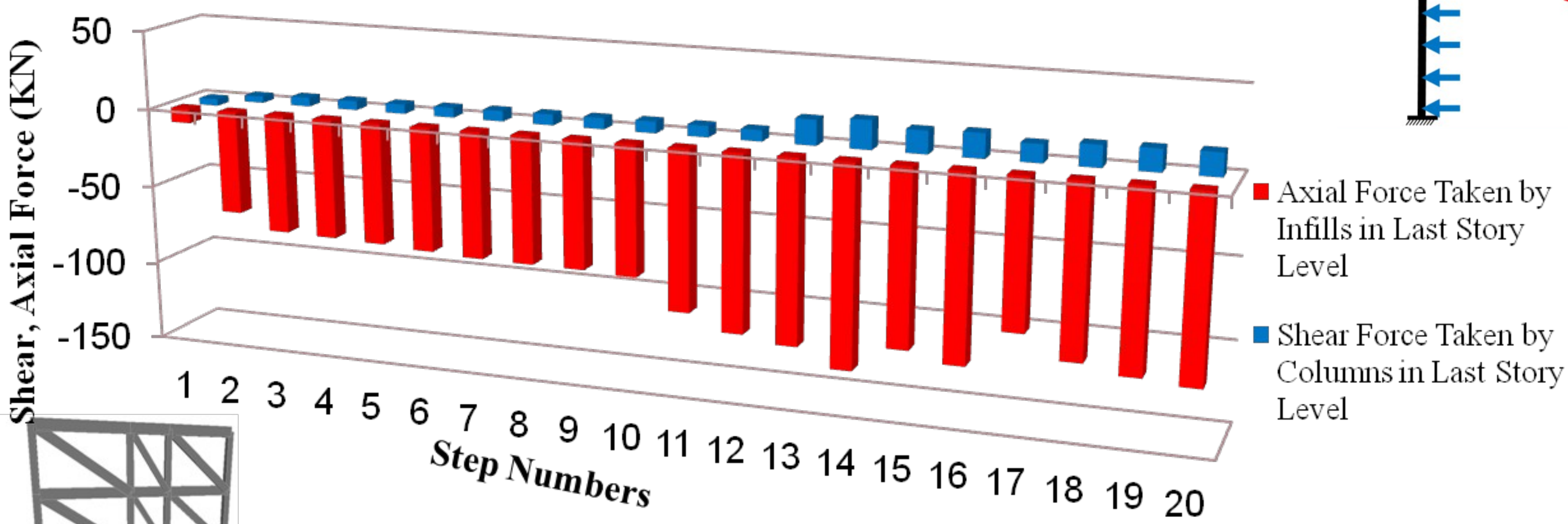
Axial Force in Infills at 1st Floor Level

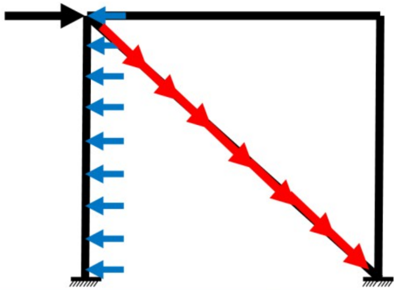
Shear Force in Columns at 1st Floor Level

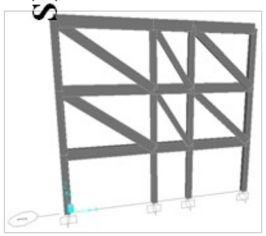


Nonlinear Pushover Analysis (Components Response)

Contribution of Infill and columns in taking shear Force at The last story

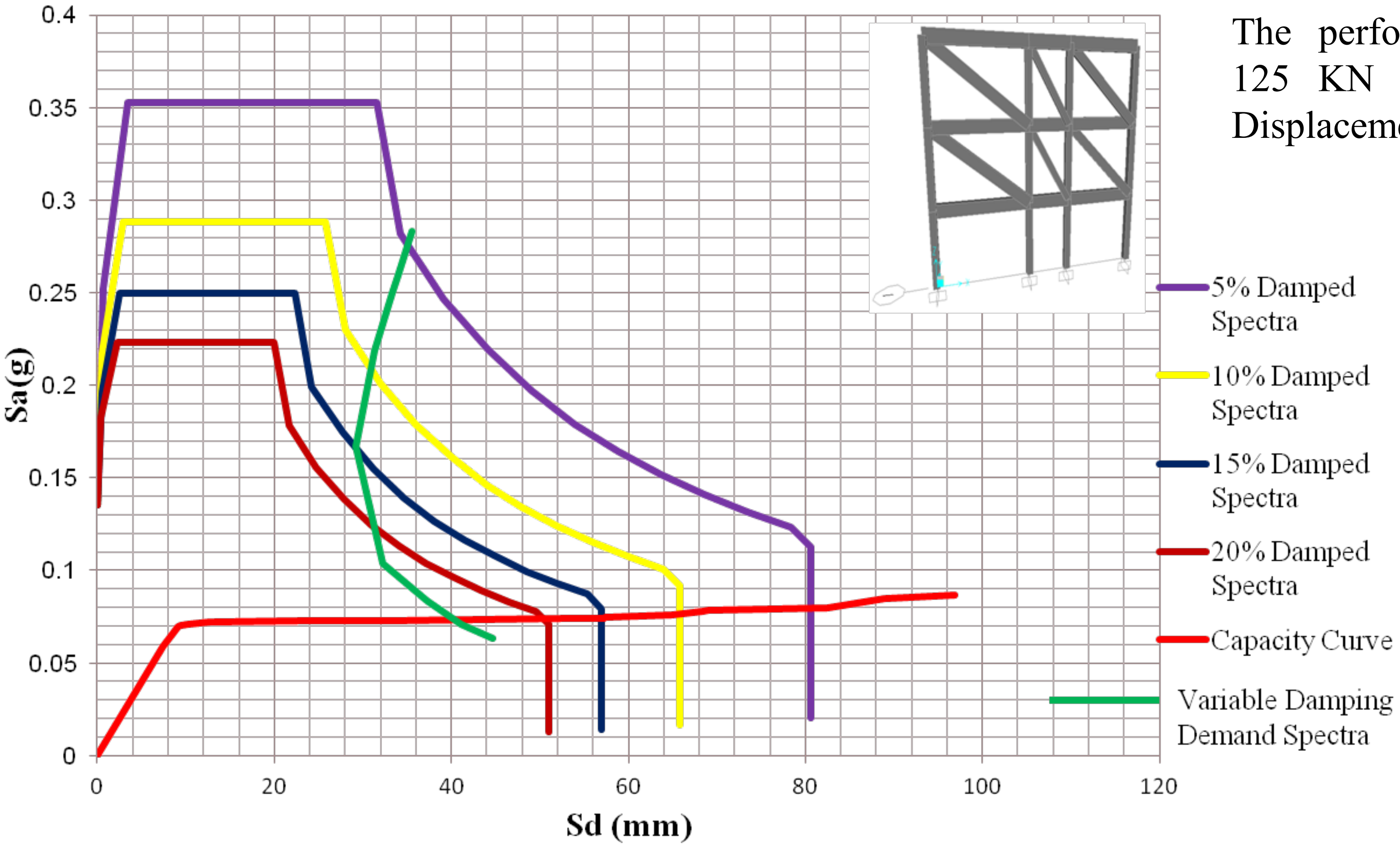






Use of Pushover Curve (ATC-40)

(System Response)



The performance point is at 125 KN Base Shear having Displacement 41 mm

Linear Static Analysis

BARE FRAMEE

Linear Static Analysis (System Response)

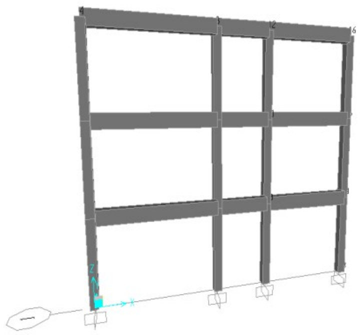
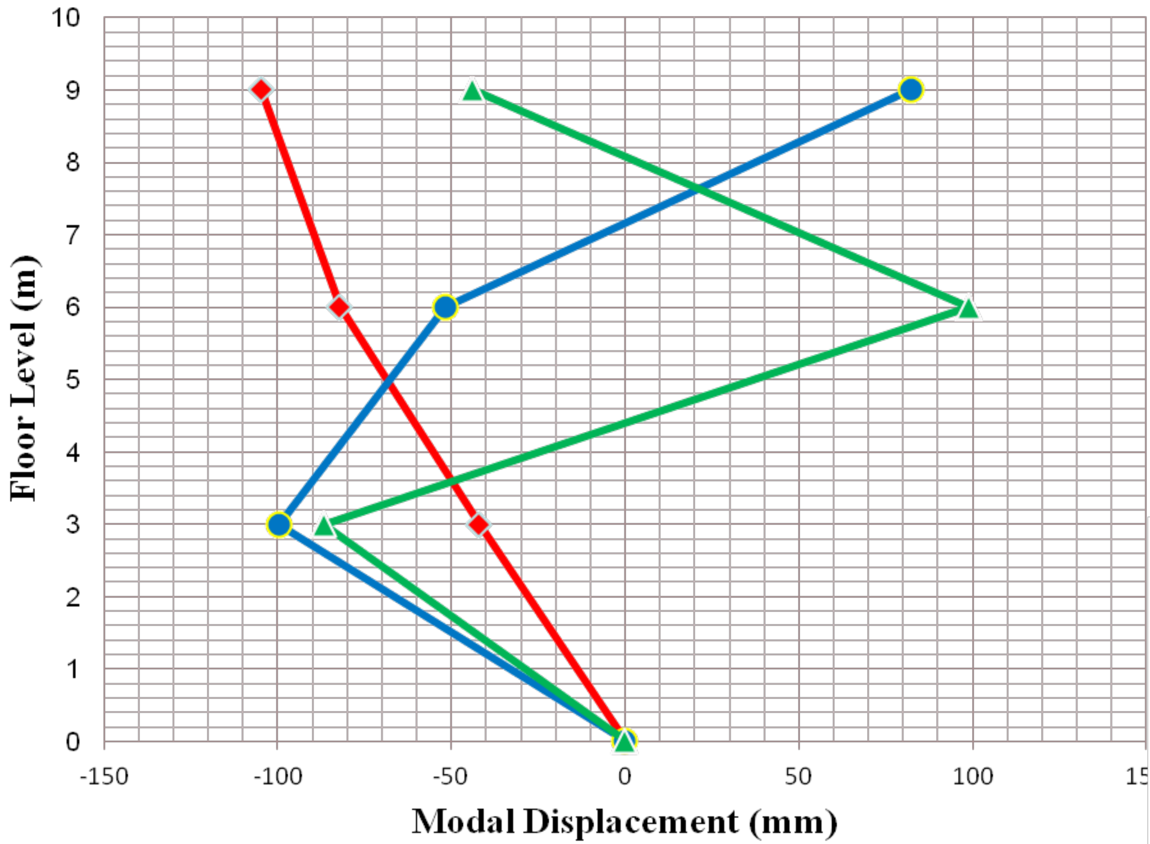


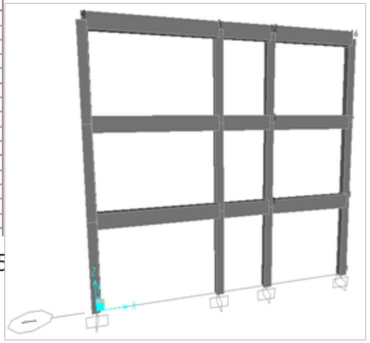
TABLE: Modal Participating Mass Ratios

OutputCase	StepType	StepNum	Period	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX	SumRY	SumRZ
Text	Text	Unitless	Sec	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
MODAL	Mode	1	0.88079	0.89568	0	3.564E-07	0.89568	0	3.564E-07	0	0.53522	0	0	0.53522	0
MODAL	Mode	2	0.30201	0.08799	0	0.0000020	0.98367	0	0.0000024	0	0.00324	0	0	0.53847	0
MODAL	Mode	3	0.19816	0.01633	0	1.412E-07	1	0	0.0000025	0	0.00064	0	0	0.5391	0

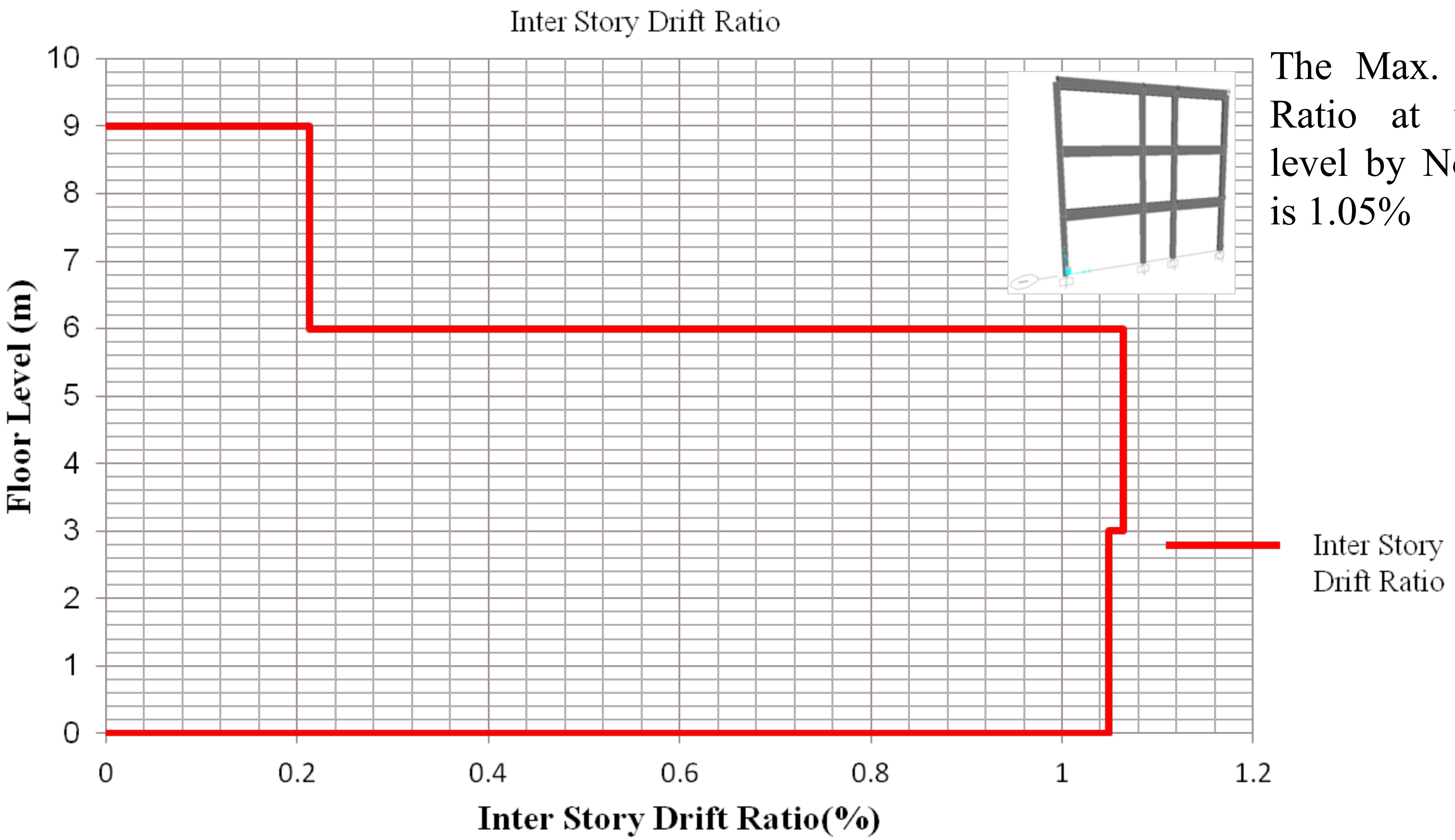
Linear Static Analysis (System Response)

The First Three Effective Modes Shapes



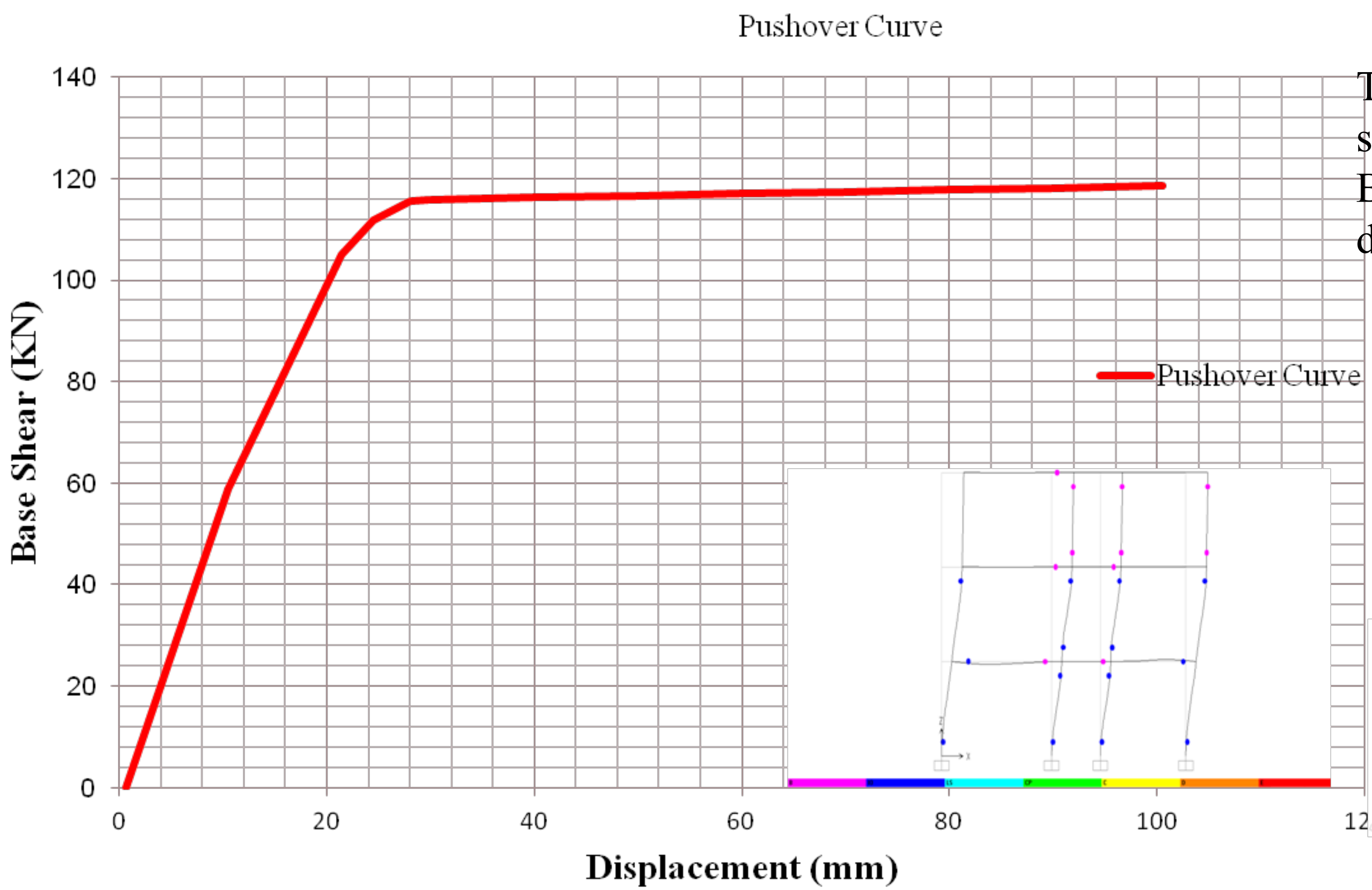


Nonlinear Pushover Analysis (System Response)

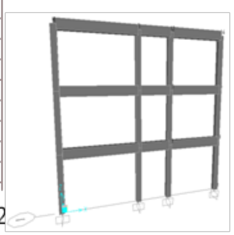


The Max. inter story Drift Ratio at the Performance level by Nonlinear Analysis is 1.05%

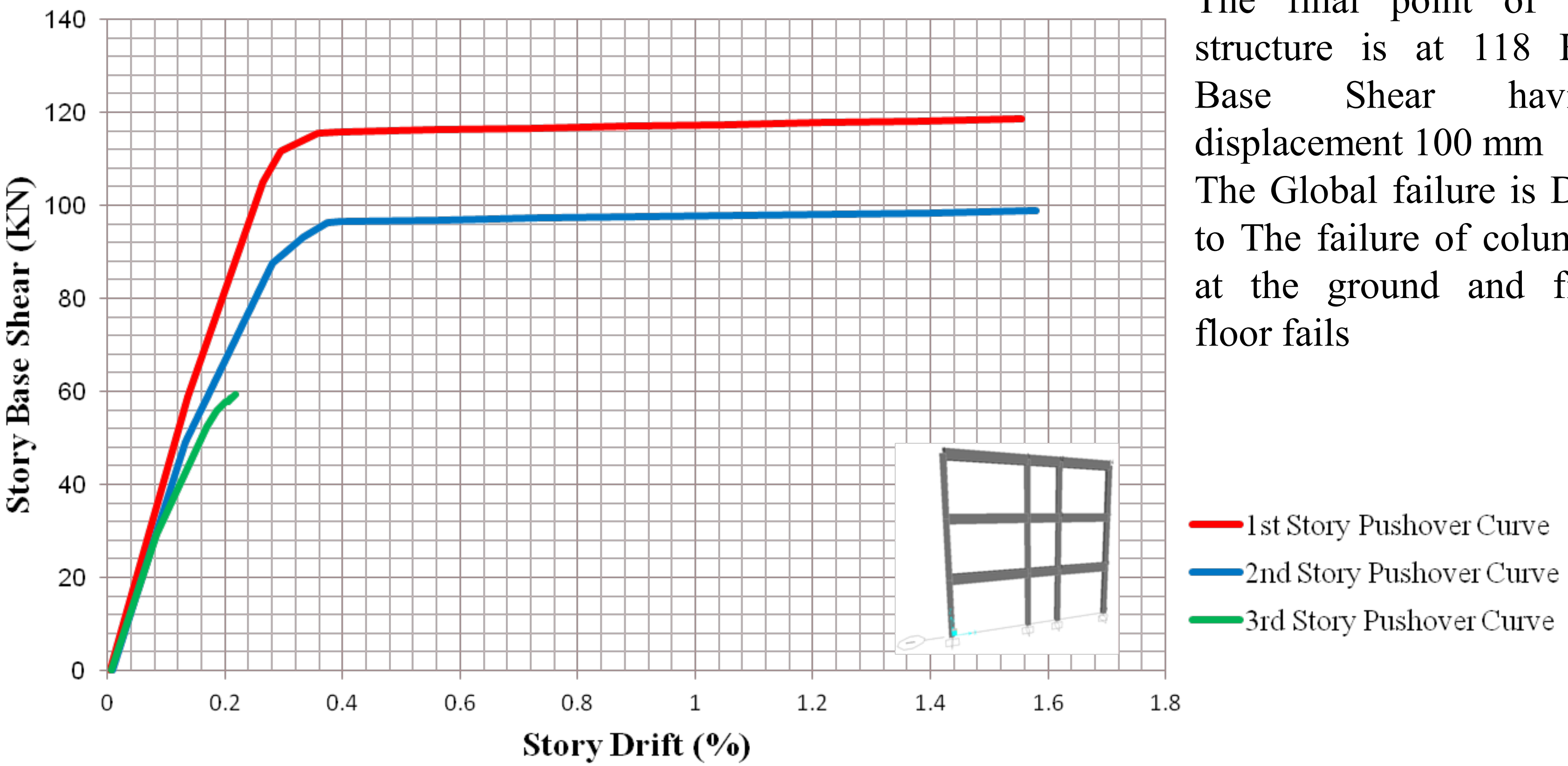
Nonlinear Pushover Analysis (System Response)



The final point of the structure is at 118 KN Base Shear having displacement 100 mm

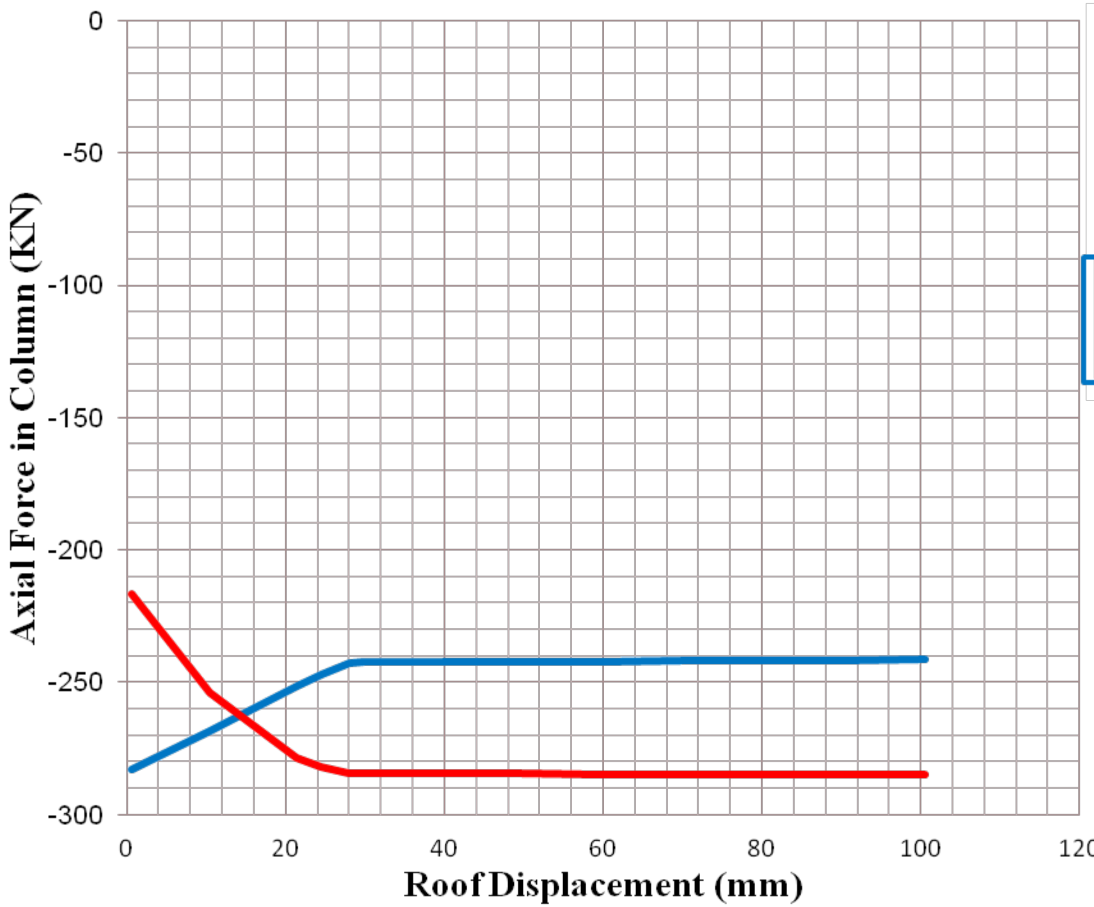


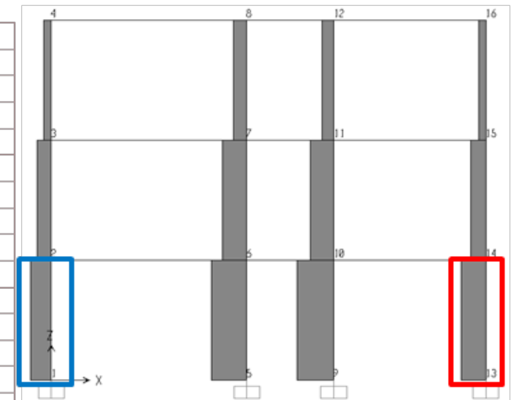
Nonlinear Pushover Analysis (System Response)



The final point of the structure is at 118 KN Base Shear having displacement 100 mm
The Global failure is Due to The failure of columns at the ground and first floor fails

Nonlinear Pushover Analysis (Components Response)





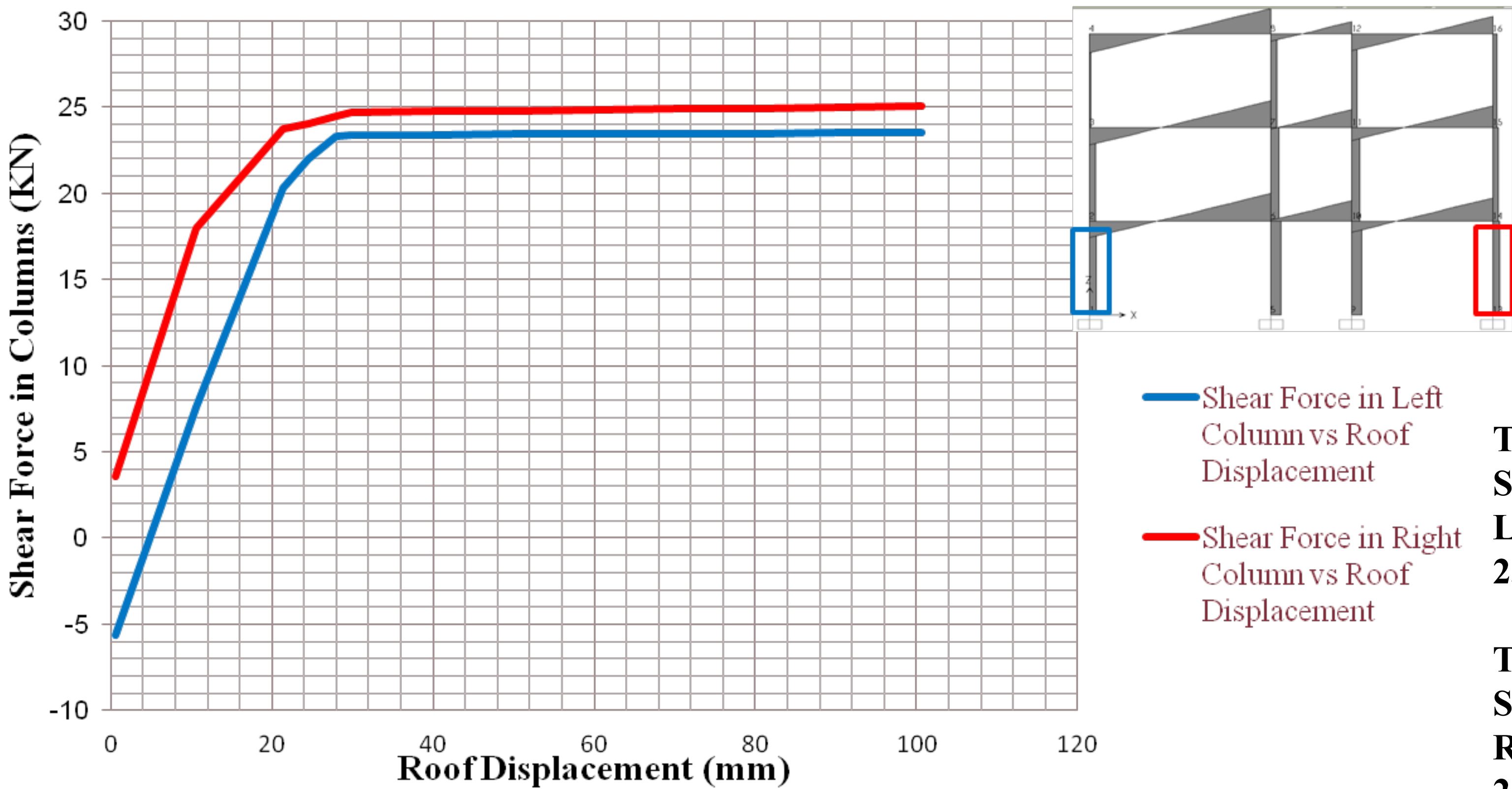
Axial Force in Left Column vs Roof Displacement

Axial Force in Right Column vs Roof Displacement

The maximum Axial force in Left Column is -241 KN

The maximum Axial Force in Right column is -216 KN

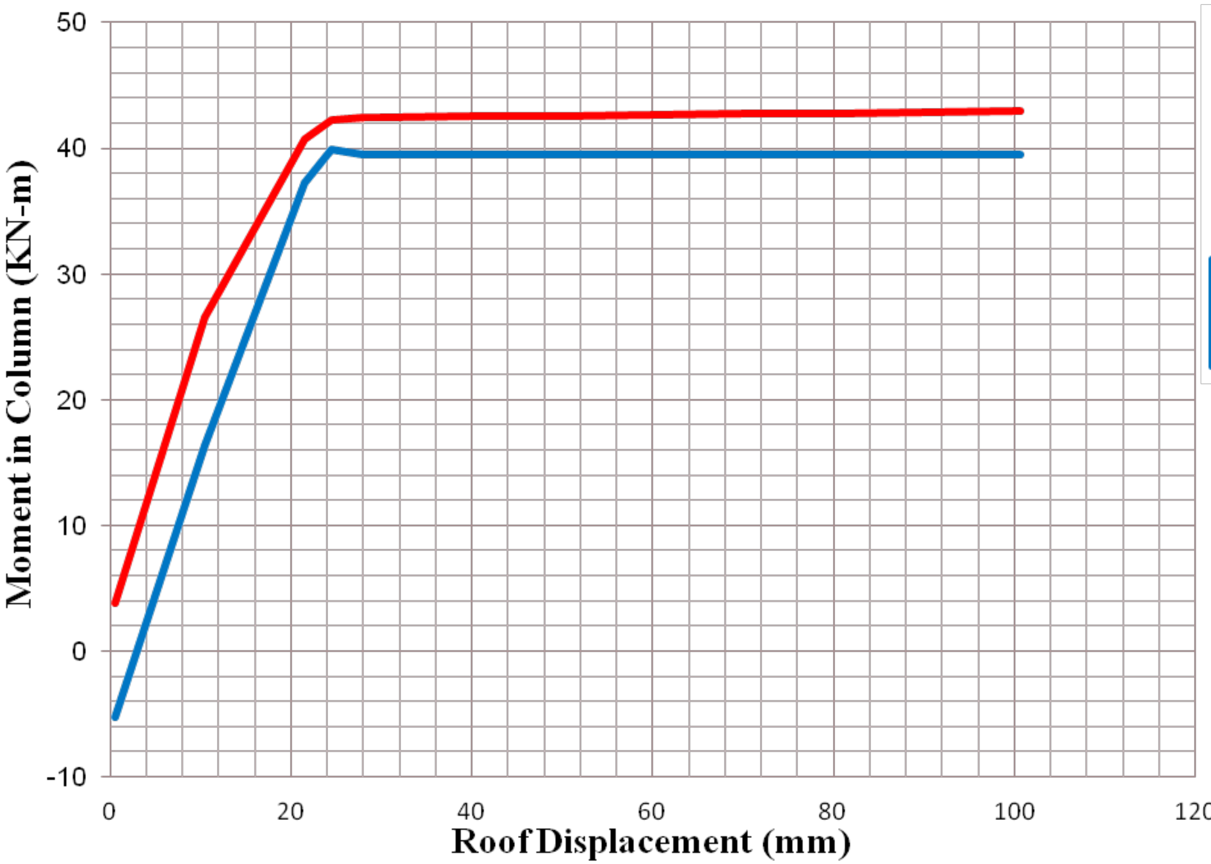
Nonlinear Pushover Analysis (Components Response)

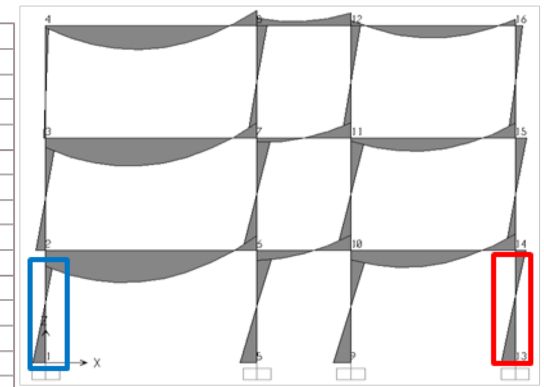


The Maximum Shear Force in Left Column is 23.5 KN

The Maximum Shear Force in Right Column is 25 KN

Nonlinear Pushover Analysis (Components Response)





Moment in Left Column Vs Roof Displacement

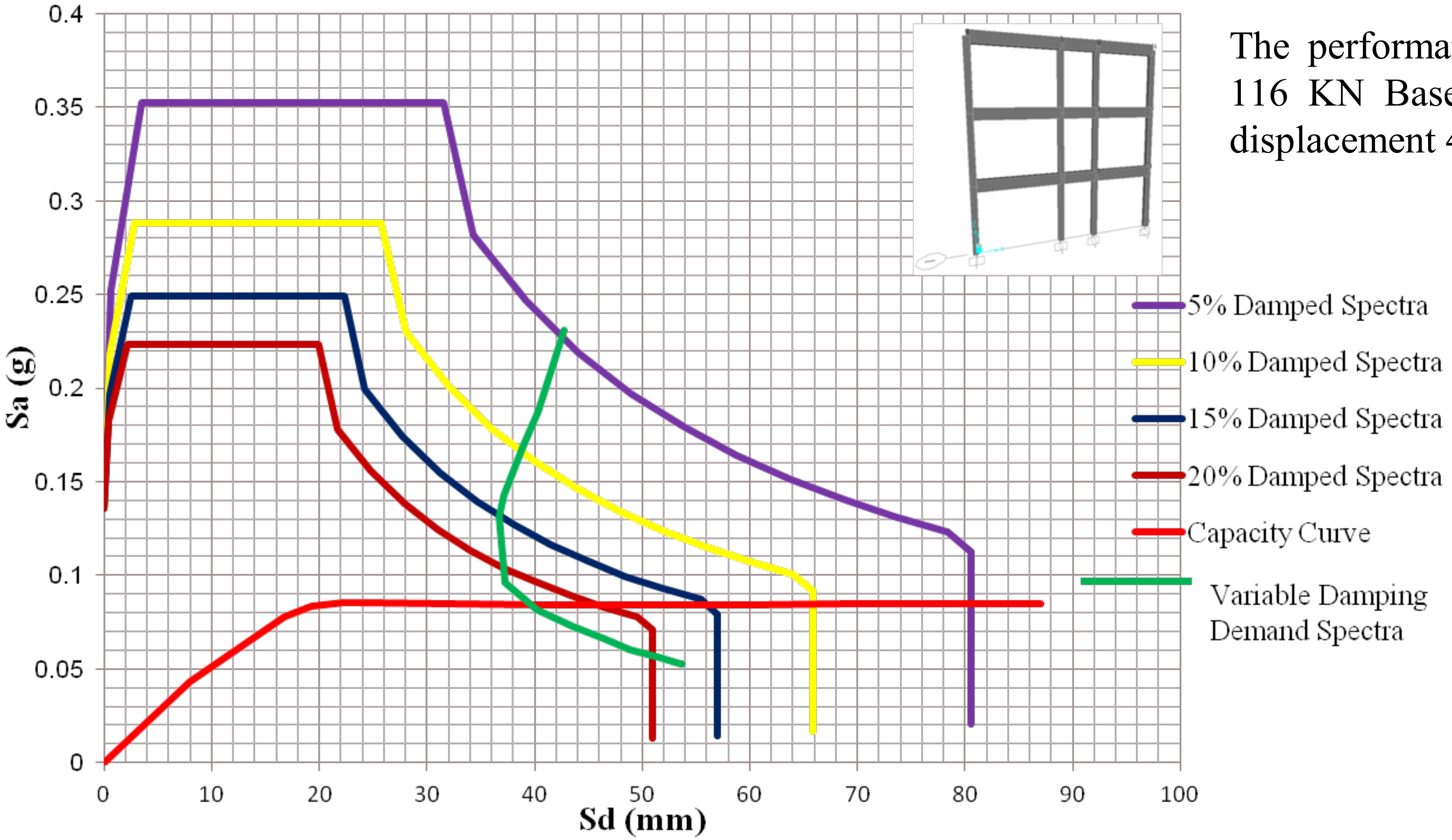
Moment in Right Column Vs Roof Displacement

The Maximum Moment in Left Column is 39.5 KN-m

The Maximum Moment in Right Column is 43 KN-m

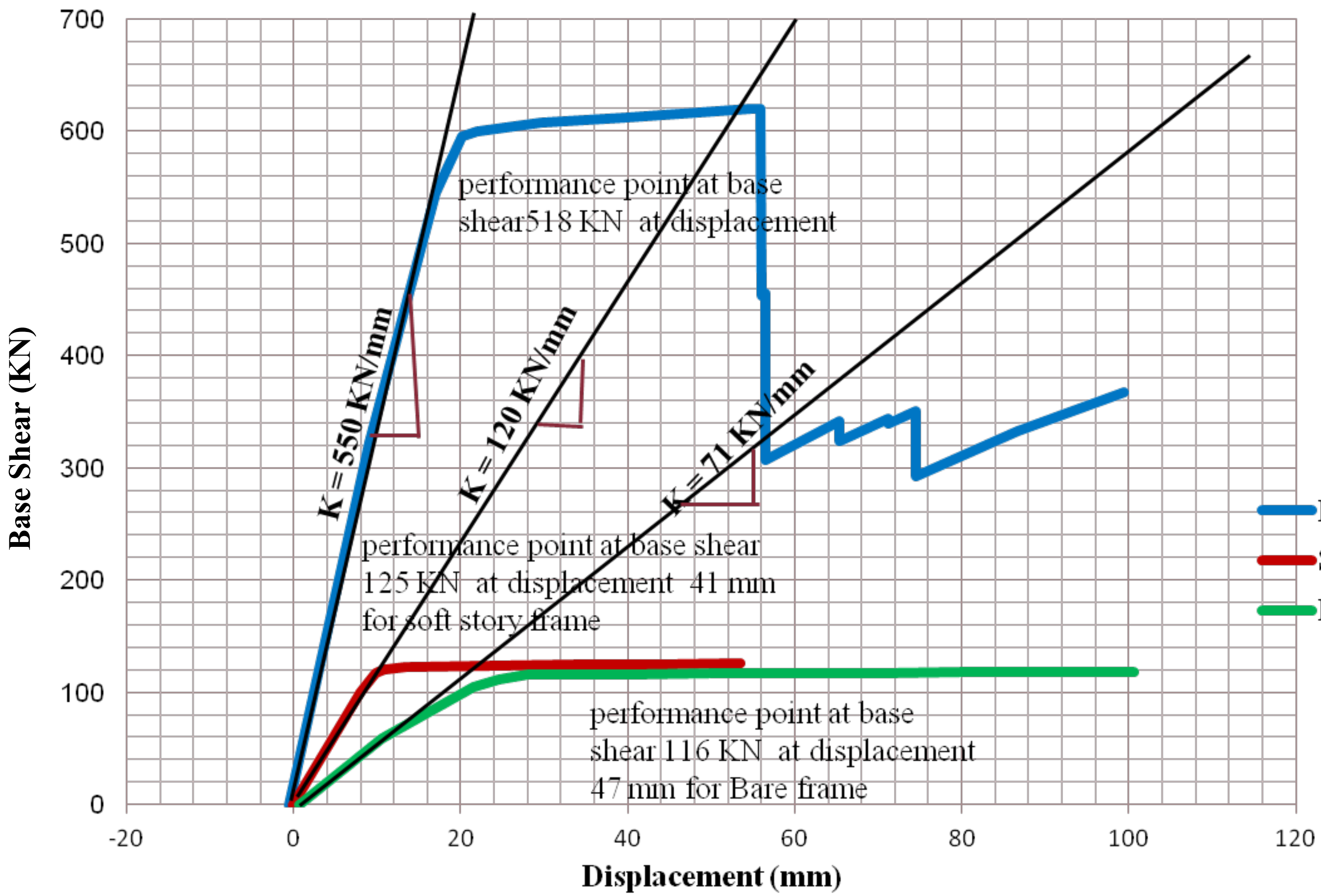
Analysis

Use of Pushover Curve (ATC-40) (System Response)



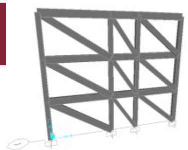
The performance Point is at 116 KN Base Shear having displacement 47 mm

Nonlinear Pushover Analysis (System Response)



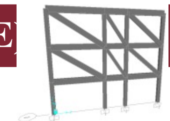
The three Frames Pushover and Demand Curves are shown together with their performance Points and stiffness

FULL INFILL FRAME



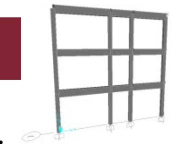
- The fundamental mode's period is 0.38 sec
- Max IDR is 0.235%
- Relative Displacement increase linearly has Range 1mm
- Stiffness is 550 KN/mm
- The Performance Point according to ATC-40 Capacity Spectrum method is at 16 mm displacement having Base Shear 518 KN
- Target Displacement according to FEAM-356 Coefficient method is 95.3 mm
- Global Ductility is 2.8 and failure of Infill and Columns Cause the overall Collapse mechanism.

(SOFT STORY FRAME)



- The fundamental mode's period is 0.72 sec
- Max IDR is 1.7%
- Because of the presence of infill at the 1st and last story the floors do not displace so much relative to each other but the Ground Floor displace abruptly relative to the First Floor having variation about 14 times
- Stiffness is 120 KN/mm
- The Performance Point according to ATC-40 Capacity Spectrum method is at 41 mm displacement having Base Shear 125 KN
- Target Displacement according to FEAM-356 Coefficient method is 117 mm
- Global Ductility is 5.4 failure of Columns at Ground Story Cause the overall Collapse mechanism.

BARE FRAME



- The fundamental mode's period is 0.90 sec
- Max IDR is 1.05%
- The Ground and first floors Displace relative to each other having variation from 10 mm to 16.5 mm
- Stiffness is 71 KN/mm
- The Performance Point according to ATC-40 Capacity Spectrum method is at 47 mm displacement having Base Shear 116 KN
- Target Displacement according to FEAM-356 Coefficient method is 160 mm
- Global Ductility is 9.6 failure of Columns and Beams Cause the overall Collapse mechanism.

DESIGN RECOMENDATION

- The infill walls has great influence in global as well as local Response of the structure, so infill walls should consider during design new building and also evaluation and Retrofitting the existing buildings.

- Great computational modeling should perform while evaluating the existing infilled RC Structures.

CONCLUSION

•URM infill walls have a significant role in the strength and ductility of RC frame structures and should be considered in both analysis and design. Globally, these walls make the structure significantly stiffer, reduce the natural period of the structure, and increase the damping coefficient

•Masonry infill walls have a complex behavior due to the properties of their materials and to the interaction mechanisms with the surrounding frame.

•.The performance of fully masonry infill walls Frames' both in 2D and 3D analysis was significantly superior to that of bare frames and soft storey frames.

•The proposed macro-model can be a useful tool in the development and calibration of simplified rules for the analysis of infilled frame structures under horizontal loadings.

SPECIAL THANKS

THANKYOU FOR YOUR ATTENTION