Chapter 1

INTRODUCTION

1.1 Background and Motivation

The Earthquake is one of the most devastating natural phenomenon. The movement of the tectonic plates relative to each other, both in direction and magnitude, leads to an accumulation of strain, both at the plate boundaries and inside the plates. This strain energy is the elastic energy that is stored due to the straining of rocks, as for elastic materials. When the strain reaches its limiting value along a weak region or at existing faults or at plate boundaries, a sudden movement or slip occurs. The tectonic events, volcanism, collapse of subterranean cavities or manmade effects may be the cause of minor or severe vibrations on the surface of the earth also called seismic motion. This unpredicted seismic motion can damage the structures.

Every year thousands of people are rendered homeless, displaced, injured and even die all over the world due to earthquakes. An earthquake is a current problem of great political and social relevance and a challenge to the structural Engineers. .

In the past, building structure have been designed without any consideration of the seismic effects. The knowledge about the earthquake, their behaviour and their effects on structures grew with time and seismic resistant design procedures have been started to be followed in the analysis and design of structures.

During the past thirty years moderate to severe earthquakes have occurred in India at intervals of 5 to 10 years. Most of the Indian building are vulnerable to seismic action even if located in areas of less seismic activities. Earthquake resistant design for new building and retrofitting measures for seismic vulnerable building are among the most important aspects for mitigating seismic hazards in earthquake prone area.

Various techniques are readily available for the seismic rehabilitation of structures. These techniques include adding of cross bracings, structural shear walls ,supplemental damping, base isolation systems, adding infill wall, jacketing of beams &columns, fibre reinforced plastic(FRP) etc. The cross bracings and structural/shear walls help reduce the drift and increase the ductility of the structures.

The base isolation system with considerable lateral flexibility help in reducing the earthquake forces transmitted to the superstructure by changing the structure's fundamental period to avoid resonance with predominant frequencies of the earthquake and reduces the floor acceleration induced by the earthquake. It is generally accepted that a base isolation building will perform better than a conventional building in moderate and strong earthquake but requires more initial investment.

Thus development of seismic resistant technique is like grace of God for the mankind. The techniques of earthquake resistant features can be applied to the new structures as well to retrofit an existing structures to ensure added safety to them.

1.2. Objectives of Project

1. Seismic evaluation of Existing RCC Framed Structure
2. Analysis of retrofitting techniques to achieve desired building performance
3. Design of base isolation
4. Comparison of the retrofitting techniques, (shear wall, base isolation & bracing system)
5. Utility of techniques

Chapter 2

LITERATURE REVIEW

2.0 Preamble:

Each new structure or existing structure poses a challenge to the engineers who have to design and build it .Each designer requires relevant guidelines as per project requirement. Some relevant literature from CED-39,IS 1893(part 1)2002 and from internet have been reviewed in this chapter

2.1Main Clause From CED-39

This part of literature review is of CED-39 some main clause for the seismic evaluation and strengthening of existing reinforced concrete buildings – guidelines.

Clause- 1.1 This document is intended to reduce the risk of death and injury that may result from the damaging effects of earthquakes on building which predate the current seismic codes [IS 1893(Part-1):2002, IS 4326:1993 and IS13920:1993] are have not been designed for earthquake forces.

Clause 2.1 Checking Original Design Details

The following details shall be checked in the original design. Any deficiency should be considered in choosing the response reduction factor R in detailed evaluation and in the retrofit design.

a) No Shear Failures — Shear capacity of frame members shall be adequate to develop the moment capacity at the ends, and shall be in accordance with provisions of IS: 13920 for shear design of beams and columns.

b) Concrete Columns – All concrete columns shall be adequately anchored from top face of pedestal of base slab to the foundation.

c) Strong Column/Weak Beam – The sum of the moment of resistance of the columns at any joint shall be at least 1.1 times the sum of the moment of resistance of the beams along each principal plane of the frame joints.

d) Beam Bars - At least two longitudinal top and two longitudinal bottom bars shall extend continuously throughout the length of each frame beam. At least 25% of the longitudinal bars located at the joints for either positive or negative moment shall be continuous throughout the length of the members.

e) Column-Bar Splices - Lap splices shall be located only in the central half of the member length. It should be proportioned as a tension splice. Hoops shall be located over the entire splice length at spacing not exceeding 150 mm centre to centre. Not more than 50 percent of the bars shall preferably be spliced at one section. If more than 50 percent of the bars are spliced at one section, the lap length shall be 1.3 L_d where L_d is the development length of bar in tension as per IS 456: 2000.

f) Beam- bar Splices - Longitudinal bars shall be spliced only if hoops are located over the entire splice length, at a spacing not exceeding 150 mm. The lap length shall not be less than the bar development length in tension. Lap splices shall not be located (a) within a joint, (b) within a distance of *2d* from joint face, and (c) within a quarter length of the member where flexural yielding may occur under the effect of earthquake forces. Not more than 50 percent of the bars shall be spliced at one section.

g) Column-Tie Spacing - The parallel legs of rectangular hoop shall be spaced not more than 300 mm centre to centre. If the length of any side of the hoop exceeds 300 mm, the provision of a crosstie should be there. Alternatively, a pair of overlapping hoops may be located within the column. The hooks shall engage peripheral longitudinal bars.

h) Stirrup Spacing—The spacing of stirrups over a length of *2d* at either end of a beam shall not exceed (a) *d/4,* or (b) 8 times the diameter of the smallest longitudinal bar; however, it need not be less than 100 mm. The first hoop shall be at a distance not exceeding 50 mm from the joint face. In case of beams vertical hoops at the same spacing as above shall also be located over a length equal to 2d on either side of a section where flexural yielding may occur under the effect of earthquake forces. Elsewhere, the beam shall have vertical hoops at a spacing not exceeding *d/2.*

i) Joint Reinforcing— Beam-column joints shall have ties spaced at or less than 150 mm.

k) Stirrup and Tie Hooks - The beam stirrups and column ties shall preferably be anchored into the member cores with hooks of 135° and 6d extension.

Clause 8.5.1.1 *RCC Jacketing Of Columns*

The procedure for reinforced concrete jacketing is:

a) The seismic demand on the columns, in terms of axial load (P) and moment *(M)* is obtained.

b) The column size and section details are estimated for *P* and *M* as determined above.

c) The existing column size and amount of reinforcement is deducted from the values obtained considering the demand.

d) The extra size of column cross-section and reinforcement is provided in the jacket.

e) The actual concrete and steel provided in the jacket is as given below:

$A_c = (3/2)\, A_c'$ and $A_s = (4/3) A_s'$

where

A_c and A_s= Actual concrete and steel to be provided in the jacket

A_c' and A_s' = Concrete and steel values obtained for the jacket after deducting the existing concrete and steel

f) The spacing of ties to be provided in the jacket in order to avoid flexural shear failure of column and provide adequate confinement to the longitudinal steel along the jacket is given as:

$$s = \frac{f_y\, d_h^2}{\sqrt{f_{ck}}\, t_j}$$

Where f_y= yield strength of steel f_{ck} = cube strength of concrete

d_h= diameter of stirrup t_j = thickness of jacket

g) Bent down bars:-In order to transfer the additional axial load from the old to the new longitudinal reinforcement, bent down bars are provided which are intermittent lap welded to bars of jacket and longitudinal bars in the existing column exposed for the purpose. Moreover, bent-down bars help in good anchorage between existing and new concrete.

h) The number of bent-down bars required is given as,

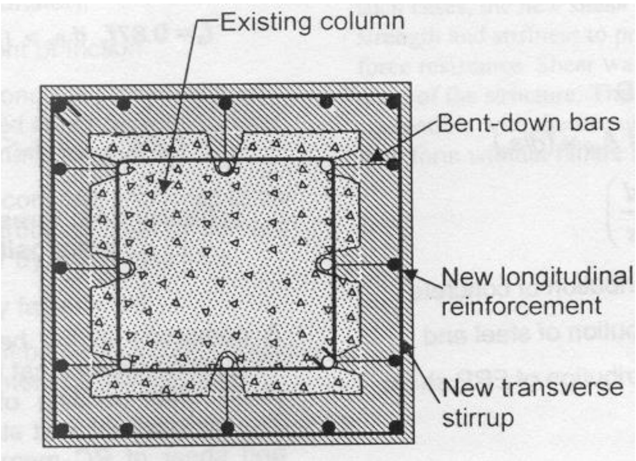
$$\frac{\Delta P}{20\, Asb + 10}$$

where ΔP = additional axial load to be transferred to the jacket reinforcement.

A_{sb}= total cross-section of the bent down bars.

h_s = width of bent-down bars.

j) If the transfer of axial load to new longitudinal steel is not critical then friction present at the interface can be relied on for the shear transfer, which can be enhanced by roughening the old surface.



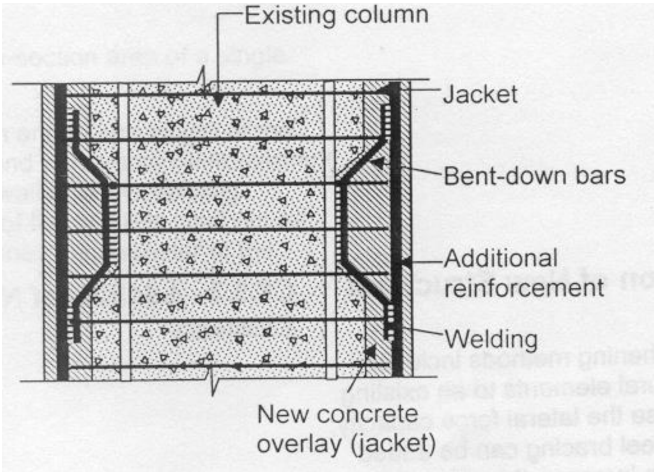


Figure No 2.1 Jacketing Of Columns

Clause:-Fibre Jacketing of A Beam

Dimensions of FRP jacket is determined assuming composite action between fibres and existing concrete. The rupture strength of FRP is used as its limiting strength.

Limit state moment capacity of FRP retrofitted member is given by:-

Ultimate flexural strength is determined based on the assumption that compressive concrete reaches a strain of 0.0035 and FRP reaches its maximum strain.

Shear strength of a beam after strengthening:-

$V = V_{con} + V_S + V_{FRP}$

where,

$V_{con}= t_c \times b \times D$

$V_S = 0.87 \times f_y \times A_{sv} \times (d/S_v)$

$V_{FRP} = A_f f_f \ (d/s)$

V_{con} is shear contribution of concrete

V_S is shear contribution of steel and

V_{FRP} is shear contribution of FRP sheet

Clause:- Addition of New Structural Elements

One of the strengthening methods includes adding new structural elements to an existing structure to increase the lateral force capacity. Shear walls and steel bracing can be added as new elements to increase the strength and stiffness of the structure.

Clause:-8.5.2.1 Addition of new reinforced concrete shear walls provides a better option of strengthening an existing structure for improved seismic performance. It adds significant strength and stiffness to framed structures. The design of shear walls shall be done as per IS 13920.

a) The shear transfer reinforcement (dowel bars), perpendicular to the shear plane, is given as,

$$A_{vf} = \frac{V_u \eta}{f_y \mu}$$

where, V_u = Allowable shear force not greater than $0.2f_{ck} A_c$ or $5.5\ A_c$ (A_c is the area of concrete section resisting shear transfer).

μ = Coefficient of friction

=1.0 for concrete placed against hardened concrete with surface intentionally roughened.

= 0.75 for concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars.

η = Efficiency factor = 0.5

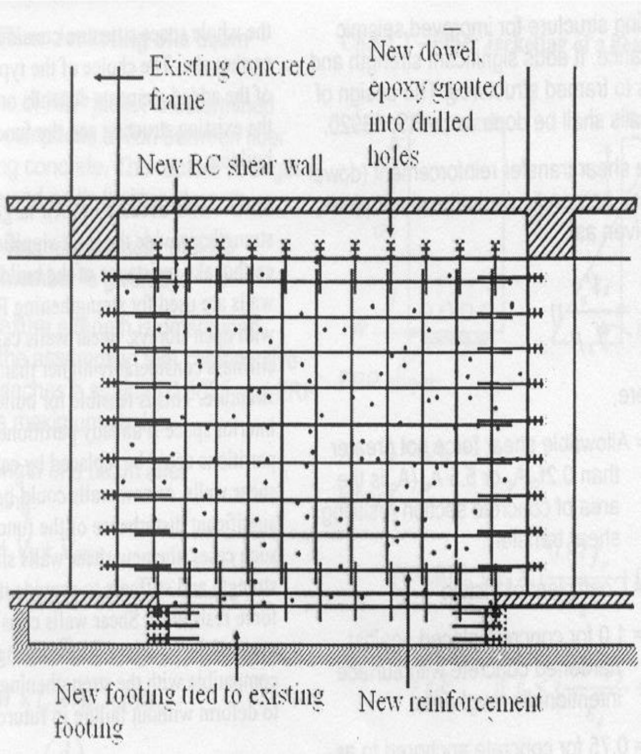


Figure No 2.2 Addition of New Structural Element

b) The number of bars required for resisting shear at the interface are given as,

$$N = A_{vf}/A_{vf}'$$

where,

A_{vf}' = cross-section area of a single bar.

c) The minimum anchorage length of the grouted-in longitudinal and transverse reinforcement of the shear wall in to the existing components of the building shall not be less than 6 times the diameter of the bars.

d) Wherever thickness of column is 250mm or less, shear wall shall encase the column by wrapping shear wall reinforcement around column after roughening RC column surface

Clause:-8.5.2.2 Steel diagonal braces can be added to existing concrete frame:-

Some of the design criteria for braces are given below:-

a) Slenderness of bracing member shall be less or equal to 2500/□f_y.
b) The width-thickness ratio of angle sections for braces shall not exceed 136/□f_y.
c) In case of Chevron (V) braces. This load shall be calculated using a minimum of yield strength P_y for the brace in tension and a maximum of 0.3 times of load capacity for the brace in compression P_{ac}.
d) The top and bottom flanges of the beam at the point of intersection of V braces shall be designed to support a lateral force equal to 2% of the beam flange strength $f_y b_f t_f$.
e) The brace connection should be adequate against out-of-plane failure and brittle fracture.

2.2 Design Criteria for Earthquake Resistant Buildings: Building Codes

Ref:-33rd conference on our world in concrete & structures: 25 – 27 august 2008, Singapore seismic isolation for medium rise reinforced concrete frame buildings

2.2.1. Conventional Method (IS 1893 (Part I): 2002))

The structures are designed to resist specified static lateral force related to the properties of structures and zone seismicity

Based on the formulae specified, an estimate for

1. The fundamental natural vibration period
2. Base shear and
3. The distribution of base shear, can be computed. Static analysis of the building for lateral forces provides including shear and overturning moments for various stories.

Assumptions

1. Fundamental natural vibration period and amplitude lasting for small duration due to impulsive ground motion.
2. Earthquake will not occur simultaneously with wing and/or maximum flood and/or maximum sea wave action.
3. Elastic modulus of materials is considered same as that for static analysis

Permissible Stresses and Load Factors:-

1. No stresses increase is allowed in the Limit State Design Method
2. Ultimate State Design Method, the yield stress of steel limited to 80% of ultimate strength or 0.2 percent proof stress whichever is smaller.
3. For Limit State Design Method, a partial safety factor can be taken as per IS 456:2000

2.2.2 Seismic Design Coefficient Methods:-

For determining the seismic design forces, the structures depend on its own dynamic characteristics and the ground motion due to earthquake.

There are two methods: -

1. Seismic Coefficient Method
2. Response Spectrum Method

2.2.2.1. Seismic Coefficient Method:-

This method is simple and may be used for simple structures where Response Spectrum Method is not warrant. In this method, the seismic forces can be computed on the basis of importance of the structures and its soil- foundation systems.

The horizontal seismic coefficient, αh , can be computed as :-

$\alpha h = \beta\ I\ \alpha 0$where:-

β= a coefficient depending upon the foundation system.

I = a factor depending upon the Importance of the structure.

$\alpha 0$ = basic horizontal seismic coefficient.

2.2.2.2. Response Spectrum Method:-

In this method, first the response acceleration coefficient for the natural vibration period and damping of the structure are required. Based on these values, the horizontal seismic coefficient, αh, can be computed as :-

$\alpha h = (Z\ I\ Sa)/(2Rg)$

where,

R = performance factor depending upon the structural Framing system.

I = a factor depending upon the Importance of the structure.

Z = Seismic Zone Factor for average acceleration spectra .

Sa/g = average acceleration coefficient based on appropriate natural periods and damping of the structure

Fundamental Period of Building:-

Fundamental period of time, T for moment resisting frames building without bracing or shear wall scan be calculated as (IS 1893(Part I): 2002))

$T = 0.075\ h^{0.75}$ or $T = 0.09h/d^{0.5}$ Where

h = height of building, in m.

d = base dimension of building at the plinth level in m. along the Considered direction of the lateral force.

2.2.2.3 Base Shear:-

Base shear, V_b, is calculated as:-

$V_b = C\ \alpha_h\ W$ Where

C = a coefficient depending upon the fundamental time period,

α_h = design seismic coefficient .

W = Dead Load (as specified in IS 875:1987 (8)) + Live Load (as defined in Table- 4 of IS code.)

2.2.2.4 Distribution of Forces along with Height of the Building:-

The distribution of forces along with height of the building can be expressed as:-

$$Q = V_b \frac{W_i h_i^{2}}{\sum_{i=1}^{i=n} W_i h_i^2}$$

where Q = lateral force at floor i

V_b = base shear

W_i = load (DL + LL (as specified in code)) of the roof or any floor i

h_i= height measured from the base of the building to the roof or any floor

n = number of story including basement

2.3. Base Isolation System:-

2.3.1Concept

i. Isolation units are the basic elements of base isolation system which provide the mentioned separation effect to a building structure.
ii. Isolation components are the connections between isolation units and other parts of the building having no separation effect of their own.
iii. There are three main forms of base isolation systems.
iv. Flat plate sliding bearings,
v. Friction pendulum sliding bearings,
vi. Elastomeric bearings.

Conventional method for Earthquake resistant design of building structures is primarily based on a ductility design concept but on major earthquake this concept has been proved unsatisfactory.

1. The desired “Strong Column Weak Beam” concept may not be realistic due to existence of walls
2. Shear failure of a column or short column effect
3. Construction difficulties especially at beam-column connections

To minimize the above problems, seismic isolation is one of the most promising alternatives. Seismic isolation may reduce the earthquake induced forces by factor of 3 to 8 from those that an elastic, conventional fixed base structure would experience.

2.3.2. Static Analysis

The static analysis procedure represents an upper bound estimate of seismic design loads and isolator displacements. This concept is a useful tool for preliminary design. IS 1893 (Part I):2002 allows modal analysis using response spectrum method for seismic Zones I, II,and III and up to 90 building height.

The statically equivalent seismic force, V:-

$V = k_{eff}\ d_i$

Wherek_{eff} = 4 $\pi^2 W/T^2 g$

d_i = 10AS_iT/ where

k_{eff}= isolator system stiffness

d_i = displacement across the isolation bearing

The maximum base isolation system shear force:-

V_{isol} = [AS_i/TB] W

whereV_{isol}= isolation base shear force

A = spectral acceleration coefficient at 5 percent damping

B = damping coefficient (Table- 6 Of FEMA)

2.3.3 Base Isolation in Real Buildings (Ref:- internet)

Base isolation has now been used in buildings in countries like Italy, Japan, New Zealand and USA. It has been in increased use since the 1980

1. San Francisco City Hall, California, U.S.
2. Salt Lake City/County Building, Utah, U.S.
3. Antifriction and Multi-Step Base Isolation.
4. Pasadena City Hall, California, U.S.
5. Oakland City Hall

Details of Oakland City Hall

- Building was severely damaged during the 1989 Loma Prieta earthquake
- Building is listed on the historic register – Retrofit had to preserve the interior architecture and the historic fabric of the building
- Both conventional fixed-base and base isolation retrofit concepts were studied
- The most economical and effective method was determined to be base isolation

Critical Construction Issues:

- Temporary lateral bracing during construction period to safeguard against possible earthquake occurrence
- Symmetric work sequence was important to reduce the possibility of torsional response in the event of an earthquake
- Vertical column displacement during jacking was limited to around 0.10 inches to prevent damage to superstructure finishes

Chapter 3

SEISMIC EVALUATION OF OLD BUILDING

3.0 Preamble:

The aim of evaluation is to assess the seismic capacity of earthquake vulnerable building or earthquake damaged, seismically deficient structure for future use . A seven -storey RCC building has been considered for this propose followed by detailed evaluation of suitable seismic resistant measure.

3.1Typical features of the building:-

Number of stories: - Seven -storey RCC apartment building

Year of construction -1990(Approx)

Location: - NCR (Delhi)/ seismic zone v

Strategies: - Seismic Retrofitting Techniques

Reference: -The existing six rooms at each floor) along with other relevant facilities, building consist of brick work with RCC columns & roof with continuous type of brick foundation.

There are no design records. It has been decided to examine the health and suitability of structure. It has been decided to perform the following inspections and health check test.

1. Compressive strength test by Rebound Hammer (Schematic)
2. Vertically plumb test. By Inclinometer
3. Visual inspection carried out to check initial Stability Assessment.

3.2 VISUAL INSPECTION: - *At first floor cracks are observed in some beams and Brick*

i.	Beams &Columns	All Rooms
ii.	Slabs/Floors	All Rooms
iii.	Load Bearing Wall	All Rooms
iv.	Foundations	Not Inspected

Table No 3.1:- Visual Inspection of an Old Building

Structure	Items	Observation	Action To Be Taken
All Rooms	Beams, Walls, Columns	Cracks In Columns	Strengthening Required.
Stairs	Beams, Walls, Slab	Sound Condition	Strengthening required.
Toilets	Beams, Walls, Slab	Sound Condition But Dampness	Dampness Treatment required

Figure No 3.1:- Cracks In Columns & Masonry Wall

Table No.3.2:- Rebound Hammer Test Reports

S. No	Location	Structure	N-Value	Compressive Strength. Mpc
1	GF (Library)	Beam	28	24
2	GF (Library)	Column	32	26
3	GF(Toilet)	Column	25	20
4	GF(LAB)	Column	32	26
5	FF(Toilet)	Column	30	25
6	FF(ROOM-26)	Beam	28	24
7	FF(ROOM24)	Column	26	21
8	FF(ROOM25)	BEAM	28	24
			AVERAGE	23.75

3.3 CORRECTIVE ACTIONS REQUIRED: -

1. Concrete jacketing- both beams and columns at the crack location
2. Any one system of the followings

a. Adding RCC shear walls from first level to top storey.

b. Steel bracing in transverse

c. Base isolation of building

RECOMMENDATION: -Structure is safe with strengthening/ Seismic Retrofitting/ modification in original

Chapter 4

CASE STUDY

4.0 Preamble:

This chapter deals with a few case studies in which the application of the most advance technological device (base isolation scheme) is employed .This proved to be a good learning opportunity and better understanding about the behaviour of the base isolation.

The information regarding suitability, effectiveness, based on the case study are very useful for further application of scheme.

4.1Base Isolation In India and NCR

After 2001 Bhuj (Gujarat) earthquake a new 300-bed Four-Storey Bhuj hospital in earthquake-prone Gujarat State, is the first building in India to use lead-rubber base isolation technology - a building protection system developed in New Zealand and increasingly used in earthquake-prone areas of the world, particularly Japan, China and the USA.

Two Single Storey Buildings:-(one school building and another shopping complex building) in newly relocated Killari town were built with rubber base isolators resting on hard ground. Both were brick masonry buildings with concrete roof. After the

4.2Base Isolation In Delhi :-

GTB ,Hospital- 500 Bedded New ward Block is a extension- Constructed on dated 18.4.2013 ,I along with my son Vikrant Saini ,studying in DCE 3rd year civil surveyed the site and met with Shri B.K.Jain (XEN) and got following information from site .

1. **Name of Zone/Circle/Division**: PWD DELHI BP Zone B-2/B-22/B-222.
2. **Area of the plot**: The building is located in the G.T.B. hospital campus. Total area of campus is about 90 Acres having, Plinth Area: 28734 Sq.m.
3. **Number of Floors**: 8 floors (G+7) eight storied RCC framed structure including service basement.
4. **Date of start**: 09.02.2007
5. **Date of** completion: 30.12.2011

4.3 Special Feature of The Buildings

New Ward Block (8 storied RCC framed structured) of size 71x43m with regular grids consisting of-148 No. columns was adopted.

The 148 all lead rubber bearings (132 nos. Type – A and 16 Nos.Type – B) below the columns to isolate super structure from foundation system.

Detail of Bearings:

Type-A (column load = 4000 KN)

- Dia of the bearing = 648mm
- Overall height of bearing = 315mm
- Dia of lead core = 110mm

Type-B (column load = 7000 KN)

- Dia of the bearing = 750mm
- Overall height of bearing = 315mm
- Dia of lead core = 140mm

Shear Columns sizes are 1000x1000mm below bearings to accommodate additional shear due to movement of the building during the earthquake.

Space Around Building As the space of 57 cm was required to be maintained around the building, so as to facilitate the movement during the earthquake

Sliding Type Cantilever Slab. A cantilever Type slab resting over the retaining wall at the periphery of the building was worked out to allow the movement during the earthquake.

Figure No 4.1Location of Isolators on columns foundation

Figure no:- 4.2 G. T. B. Hospital Shahadra Delhi front elevation

Figure no:- 4.3 Position of Isolators Between Base and Columns

4.4List of suppliers of Isolation system in Delhi:-

1. Alan Wilson (Chief Executive)/Robinson Seismic Ltd Phone: +64 4 569 7840/a.wilson@robinsonseismic.com

2. Sudha Palit (Senior Trade Development Manager) New Zealand Trade & Enterprise / NZ High Commission, New Delhi Phone: +91 11 2688 3170/sudha.palit@nzte.govt.nz

3. Digger Seismic Solutions, A 80,81 Vishal Enclave ,Tagore Garden, Extension, New Delhi, India, M: 931161992 (Raman Dogra),F: +91125273099
   Email: diggeotech@gmail.com

Table No 4.1:- DIS Properties

Dynamic Isolation Systems

Isolator Engineering Properties

Metric Units

DEVICE SIZE				MOUNTING PLATE DIMENSIONS					
Isolator Diameter, D_I (mm)	Isolator Height, H (mm)	Number of Rubber Layers, N	Lead Diameter D_L (mm)	L (mm)	t (mm)	Hole Qty.	Hole Ø (mm)	A (mm)	B (mm)
305	125-280	4-14	0-100	355	25	4	27	50	-
355	150-305	5-16	0-100	405	25	4	27	50	-
405	175-330	6-20	0-125	455	25	4	27	50	-
455	175-355	6-20	0-125	510	25	4	27	50	-
520	205-380	8-24	0-180	570	25	8	27	50	50
570	205-380	8-24	0-180	620	25	8	27	50	50
650	205-380	8-24	0-205	700	32	8	27	50	50
700	205-430	8-30	0-205	750	32	8	33	65	75
750	230-455	8-30	0-230	800	32	8	33	65	75
800	230-510	8-33	0-230	850	32	8	33	65	75
850	230-535	8-35	0-255	900	38	12	33	65	95
900	255-560	9-37	0-255	955	38	12	33	65	95
950	255-585	10-40	0-280	1005	38	12	33	65	95
1000	280-635	11-40	0-280	1055	38	12	40	75	115
1050	305-660	12-45	0-305	1105	44	12	40	75	115
1160	330-760	14-45	0-330	1205	44	12	40	75	115
1260	355-760	16-45	0-355	1335	44	16	40	75	115
1360	405-760	18-45	0-380	1435	51	16	40	75	115
1450	430-760	20-45	0-405	1525	51	20	40	75	115
1550	455-760	22-45	0-405	1625	51	20	40	75	115

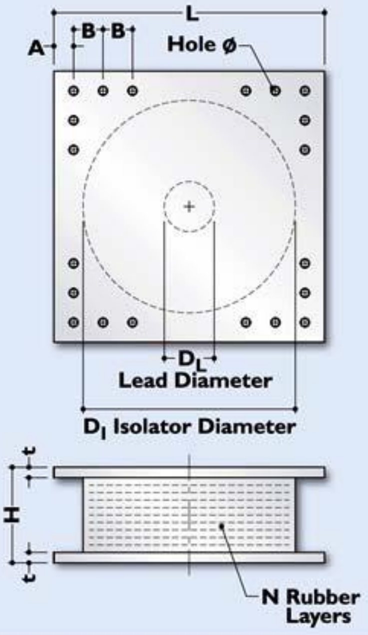
(1) The axial load capacities provided correspond to maxiumum displacements based on design limits of 250% rubber shear strain or 2/3 the isolator diameter. An isolator's actual displacement and load capacity are dependent on the rubber modulus and number of rubber layers.

(2) Rubber Shear Moduli (G) are available from 0.38 N/mm^2 to 0.70 N/mm^2.

(3) Elastic Stiffness (Ke) for analytical modeling may be taken as 10-times the yielded stiffness (Kd).

(4) Kd range shown in table is typical for most projects. If needed for specific projects, Kd values up to three times the maximum shown in the range can be achieved by limiting the displacement capacity to 2/3 of the shown value.

Isolator Diameter, D_I (mm)	DESIGN PROPERTIES			Maximum Displacement, D_{max}(mm)	Axial Load Capacity P_{max}(kN)
	Yielded Stiffness, K_d(kN/mm)	Characteristic Strength Q_d(kN)	Compression Stiffness, K_v(kN/mm)		
305	0.2-0.4	0-65	>50	150	450
355	0.2-0.4	0-65	>100	150	700
405	0.3-0.5	0-110	>100	200	900
455	0.3-0.7	0-110	>100	250	1,150
520	0.4-0.7	0-180	>200	300	1,350
570	0.5-0.9	0-180	>500	360	1,800
650	0.5-1.1	0-220	>700	410	2,700
700	0.5-1.4	0-220	>800	460	3,100
750	0.7-1.6	0-265	>900	460	3,600
800	0.7-1.6	0-265	>1,000	510	4,000
850	0.7-1.8	0-355	>1,200	560	4,900
900	0.7-1.9	0-355	>1,400	560	5,800
950	0.7-2.0	0-490	>1,800	610	6,700
1000	0.8-2.0	0-490	>1,900	660	7,600
1050	0.9-2.1	0-580	>2,100	710	8,500
1160	1.1-2.1	0-665	>2,800	760	13,800
1260	1.2-2.3	0-755	>3,700	810	20,500
1360	1.4-2.5	0-890	>5,100	860	27,600
1450	1.6-2.5	0-1,025	>5,300	910	33,400
1550	1.8-2.5	0-1,025	>6,500	910	40,000



TERMS AND SYMBOLS :-

Elastic Stiffness, Ke: This is the initial stiffness of the isolator, typically at less than one inch displacement. Its value is dominated by the lead core size and is important in controlling the response to service loads such as wind.

Yielded Stiffness, Kd or K2: This is the secondary stiffness of the isolator and is a function of the modules, total height and area of the rubber.

Hysteretic Strength, Qd: This is the force axis intercept of the isolator hysteresis loop. This parameter relates to damping and isolator response to service loads.

Keff (Effective Stiffness): This is the isolator force divided by the displacement. This is a displacement-dependent quantity.

Yield Force, Fy: The yield force is the point in the model at which the initial stiffness changes to secondary stiffness. In reality, there is a smooth transition from one stiffness to the other, rather than a well-defined point. This value is mainly used in analytical modelling.

Energy Dissipated per Cycle, EDC: This is the area of the hysteresis loop. This value is a measure if the damping of the isolator.

Vertical Stiffness (Kv): This is the vertical stiffness of the isolator.

DBE (Design Basis Earthquake): DBE represents the ground motion that has a 10% chance of being exceeded in 50 years.

MCE (Maximum Considered Earthquake): MCE is defined as the ground motion that has a 2% probability of being exceeded in 50 years.

Chapter 5

FORMULATION OF PROBLEM

5.0 Preamble:

Special techniques are required to design buildings so that they remain practically undamaged even in a severe earthquakes. The cost depending on type of technique used shear wall/ X-bracing/ base isolation system are taken for comparative analysis

There are no present guidelines or code of practice available in the country for base isolation design. Assumption and statement of problem has described in this chapter.

5.1Description of Problem

The existing six rooms seven storied residential building along with other relevant facilities consists of brick work with RCC columns & roof with continuous type of brick foundation.

There are no design records. It has been decided to examine the health and suitability of residential structure for next 50 years.

Length=1.5x width and critical direction is taken short side only.

Forces in X direction (short direction is taken for compression for E.Q. forces only)

The building is analyzed for the following system

1. Framed Structure with adding a Shear wall.
2. Framed Structure with adding X Bracing.
3. Framed Structure with Base Isolation system.

5.2.0. Base isolation, shear wall& Bracing Analysis:-

PGA= 0.36g   Damping=20 %

Time Separation=2.5 Second.

Response Spectrum method used for Analysis

Lead core laminated rubber isolators uses

5.2.1.0. shear wall

i. Size of shear 0.300x3.000 provided on both face on short direction.
ii. The design forces as per IS 1893(Part-1)
iii. R.C.C building frame with shear wall with symmetrical configuration
iv. Assumed that it will be the part of the lateral force resisting system of the structure

5.2.2 Bracing system:-

i. Steel bracing inserted in frame to provide lateral stiffness only
ii. Lateral Drift% and lateral force are taken as controlling factors.
iii. Channel size
iv. Vertical load is taken by framed

Chapter 6

DESIGN OF BASE ISOLATERS

6.0 Preamble:

There are at present no guidelines or code of practice available in the country for base isolation design. In this chapter the isolators are designed for 250&400 MT load carrying capacity by the provision of UBC-1997,FEMA-356 and IS -1893 (Part I): 2002 .

6.1 Base Isolator Design By DIS Table

The performance criteria for the isolators system used in problem:-

1. seismic load, building weight = 450 tons (GTB HOSPITAL)
2. Total design displacement not to exceed 350mm
3. Elastic base shear not exced0.65
4. Inter story drift ratio above isolation not exceed 0.01mm

Performance of isolators taken:-

i. Isolator size = 650mm x 380mm (Dia. & Height)
ii. Lead core = 130mm
iii. Soft rubber G = 0.65 MPA
iv. Effective period = 1.5 second
v. The equivalent viscous damping Range 32%(at 50mm displacement)
vi. The equivalent viscous damping Range 13%(at 400mm displacement)

6.2 Base Isolator Design By UBC1997& FEMA

Location : Delhi , India Seismic Zone - 3 (UBC)

Step 1 : From Table 16- I

Seismic zone factor Z = 0.3

Step 2 : From Table 16- J

Soil Profile Types = SD (Stiff Soil Profile)

Step 3 : From Table 16- U

Seismic Source Type A

(Faults that are capable of producing large magnitude events and that have a high rate of seismic activity)

Maximum moment Magnitude ,M $\geq$ 7.0 ,Slip rate SR $\geq$ 5

Step 4 : From Table 16- S & Table 16- T

Near Source Factor Na =1 (closest distance to seismic Source $\geq$ 10 km)

Near Source Factor Nv =1 (closest distance to seismic Source $\geq$ 15 km)

Step 5 : From Table - A-16-D

Design Basis Earthquake shaking intensity , $Z * N_V$ = 0.3*1= 0.3

Maximum capable earthquake Response Coefficent , M_M = 1.50

Step 6 : From Table 16- R & Table 16- Q

Seismic Coefficient $C_V = C_{VD}$ = 0.54

(Soil profile type = SD , Seismic zone factor Z = 0.3)

Seismic Coefficient $C_A = C_{AD}$ = 0.36

(Soil profile type = SD , Seismic zone factor Z = 0.3)

Step 7 : From Table - A-16-G & Table - A-16-F

Seismic Coefficient ,C_{VM}

Maximum capable E.Q shaking Intensity MM*Z*Nv = 1.5*0.3*1 = 0.45

For $M_M Z N_v \geq 0.40$ for Soil profile type SD

C_{VM} = 1.6*MM*Z*Nv =1.6*.45=0.72

Seismic Coefficient ,C_{AM}

Maximum capable E.Q shaking Intensity MM*Z*Na = 1.5*0.3*1 = 0.45

For $M_M Z N_a \geq 0.40$ for Soil profile type SD

C_{AM} = 1.1*MM*Z*Na =1.1*.45=0.495

Step 8 : From Table 16- A-16-E

Basic Structural system – moment resisting frame system

Lateral Force Resisting System – (SMRF)- concrete -R1 -2

Step 9 : For Design Purposes ,15% Damping is assumed Therefore from Table A-16-C

Damping Coefficient BD = BM =1.35

Step 10 : Target time period TD = 2.5 sec

Two different high damping compounds which will be denoted

A (soft) and B (hard)

GA (Tensional stiffness) = 0.4 MPa βA (Effective damping) = 0.08

GB (Tensional stiffness) = 1 MPa , βB (Effective damping) = 0.15

Total Load = 10x25+2x400=3300 MT on base

Type and No. of Bearing Stiffness to be design.

Type A . Ten (10) at 250 tons:-

TD = 2π (W/KDmin g)^0.5

TD = 2.5 sec ,W = 250*1000 kg

K^{A}_{H} =250*1000*(2π/2.5)^2

=1577536 N/m

K^{A}_{H} =1.58 MN/m

Type B .Two (02) at 400 tons :-

TD = 2π (W/KDmin g)^0.5

TD = 2.5 sec ,W = 400*1000 kg

K^{B}_{H} =400*1000*(2π/2.5)^2

=2524057.6N/m

K B_H =2.53 MN/mc

DD = g/4π^2 CvD TD/ BD

= (9.81*0.54*2.5/(4π^2) (1.35) =0.248

With Y =1.5 take t_r =200mm

K^{A}_{H} = G_A A / t_r

=0.4*A/0.2

K^{A}_{H} =1.58

A = 0.79 m^2 ,3.14* Φ^2 /4 =A

0.79 =3.14* Φ^2 /4 , Φ =1m A=0.785m^2

Then

P^A = (250*10000/0.785) = 3.18MPa

P^B = (400*10000/0.785) = 5.09MPa

Elastic base shear from code

VS = K_H D/R_{W1} CS= VS/W = 65.15*0.2/2 =6.515MN

= 6.515MN/6.334.17*10^7

= 10.28%

Thus the sizes of isolators taken for given load are sufficient and may be used .

Chapter 7

ANALYSIS OF FRAMED BUILDING

7.0 Preamble:

The building has been analysed and results were tabulated using software ETABS 9.7.4 trial version . Though there are many design software available for analysis. ETABS is a full-featured program that can be used for the simplest problems or the most complex projects.

7.1 Description of Building

A seven story building 2x3 bays in plane 6mx6m each with 3.6m height. size of columns 300x600mmeach and beams 300x450mm

A RCC with Beam Size 300x450 and Columns 300x600

The design forces as per IS 1893(Part-1):2002 and IS 456:2000

Dead load slab = 3.75KN/M2 , Live load = 2.5 KN/M2& Wall load = 18KN/M2

L= 3 BAYX6.0M =18.0M ,W = 2X6= 12.0M H=7X3.6=25.2M

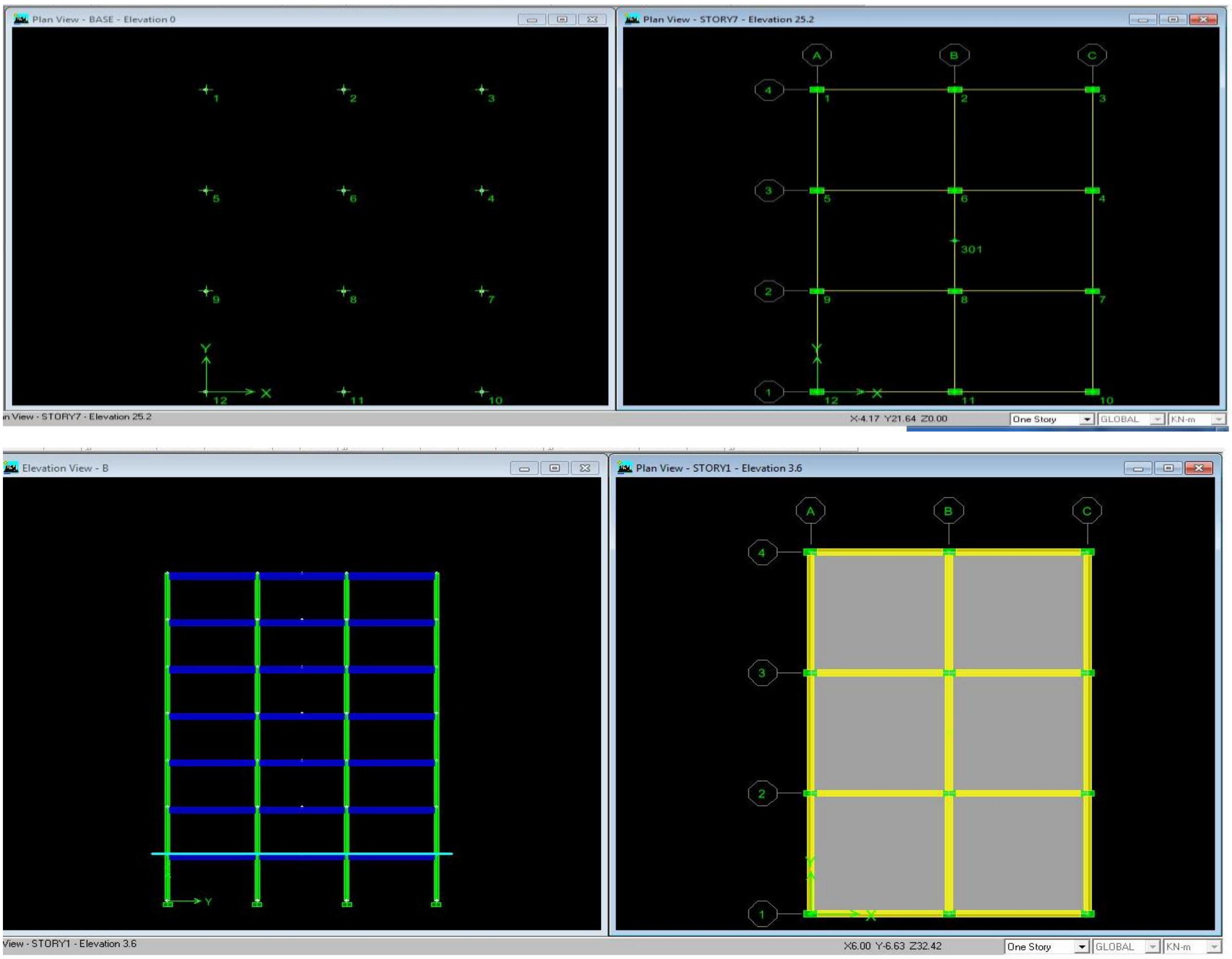


Figure No 7.1:- Plane & Elevation of Existing Building To Be Retrofitted

Table No7.1:- Results From Seven Stored Existing Building

Design loads (Unit KNM) Design load combination taken:-								
DCON 2=1.5(DL+LL), DCON3= 1.2(DL+LL+Eqx), DCON4=1.2(DL+LL-EQX)								
Result Noted for story :-			Base story, 4rth story& Top story					
Base story		Axial =33091KN		Base shear=384KN		Base Moment=33942KNM		
4rt story		Axial =14262KN		Base shear=345KN		Base Moment=163255KNM		
7rt story		Axial =3007KN		Base shear=127KN		Base Moment=27230KNM		
Story	Load	Loc	P	VX	VY	T	MX	MY
STORY7	DCON2	Top	2586.38	0	0	0	23439.375	-15464.3
STORY7	DCON2	Bottom	3007.58	0	0	0	27230.175	-17991.5
STORY7	DCON3	Top	2069.1	-127.14	0	1144.286	18751.5	-12371.4
STORY7	DCON3	Bottom	2406.06	-127.14	0	1144.286	21784.14	-14850.9
STORY7	DCON4	Top	2069.1	127.14	0	-1144.29	18751.5	-12371.4
STORY7	DCON4	Bottom	2406.06	127.14	0	-1144.29	21784.14	-13935.4
STORY4	DCON2	Top	17829.9	0	0	0	160631.1	-106925
STORY4	DCON2	Bottom	18121.5	0	0	0	163255.5	-108675
STORY4	DCON3	Top	14263.92	-345.19	0	3106.696	128504.88	-87902.3
STORY4	DCON3	Bottom	14497.2	-345.19	0	3106.696	130604.4	-90544.7
STORY4	DCON4	Top	14263.92	345.19	0	-3106.7	128504.88	-83178.3
STORY4	DCON4	Bottom	14497.2	345.19	0	-3106.7	130604.4	-83335.3
STORY1	DCON2	Top	32799.82	0	0	0	296656.425	-196313
STORY1	DCON2	Bottom	33091.42	0	0	0	299280.825	-198063
STORY1	DCON3	Top	26239.86	-384.83	0	3463.498	237325.14	-163365
STORY1	DCON3	Bottom	26473.14	-384.83	0	3463.498	239424.66	-166150
STORY1	DCON4	Top	26239.86	384.83	0	-3463.5	237325.14	-150736
STORY1	DCON4	Bottom	26473.14	384.83	0	-3463.5	239424.66	-150750

Table No 7.2:- Diaphragm Displacement Existing Building

(1) STOREY DIAPHRAGM DISPLACEMENT UNIY MM (SIMPLE FRAME)			
Story	Diaphragm	Load	Drift UX(mm)
STOREY7	D1	EQX	27.091
STOREY6	D1	EQX	24.8967
STOREY5	D1	EQX	21.5635
STOREY4	D1	EQX	17.2799
STOREY3	D1	EQX	12.3809
STOREY2	D1	EQX	7.2595
STOREY1	D1	EQX	2.5564

Table No7.3:- STOREY SHEAR EXISTING BUILDING

(2) STOREY SHEAR UNIT KN/M (SIMPLE FRAME)			
Storey	**Load**	**Loc**	**Story shear (VX)**
STOREY7	EQX	Top	-105.95
STOREY7	EQX	Bottom	-105.95
STOREY6	EQX	Top	-190.91
STOREY6	EQX	Bottom	-190.91
STOREY5	EQX	Top	-249.9
STOREY5	EQX	Bottom	-249.9
STOREY4	EQX	Top	-287.66
STOREY4	EQX	Bottom	-287.66
STOREY3	EQX	Top	-308.9
STOREY3	EQX	Bottom	-308.9
STOREY2	EQX	Top	-318.33
STOREY2	EQX	Bottom	-318.33
STOREY1	EQX	Top	-320.69
STOREY1	EQX	Bottom	-320.69
STOREY1	EQX	Bottom	-320.69

The Values for further design of Isolators:-

The flexible pads are called base-isolators. The main feature of the base isolation technology is that it introduces flexibility in the structure. The isolators are designed to absorb energy and thus add damping to the system

Vertical support Reactions for design of Isolates:-

1) 2500 KN (250TONS)

2) 4000KN (400TONS)

1 For design the Base isolators, Shear walls, X-Bracings loads may be rounded up to next higher in multiple of 50 units

2 Axial load=33091/16= 207 say 250MT may be taken for design of isolators

3 Moments for one shear wall=239424/2=119712 say120000KNM(12000TonM)

4Base shear 384/2= 192KN say 200KN(20tons)

Chapter 8

RETROFITTED STRUCTURE

8.0 Preamble: In this chapter a detailed analysis has been done for the devices shear wall, base isolators and X- bracings. Furthur the results were tabulated and corresponding graphs were plotted .RCC Shear Wall and X-bracings are provided in short direction alternatively..

Base Isolators were provided Under Each Columns for retrofitting.

8.1CASE (1):- RETROFITTED WITH SHEAR WALL

RCC Shear Wall 300x2000 Provided Both Side In X-Direction

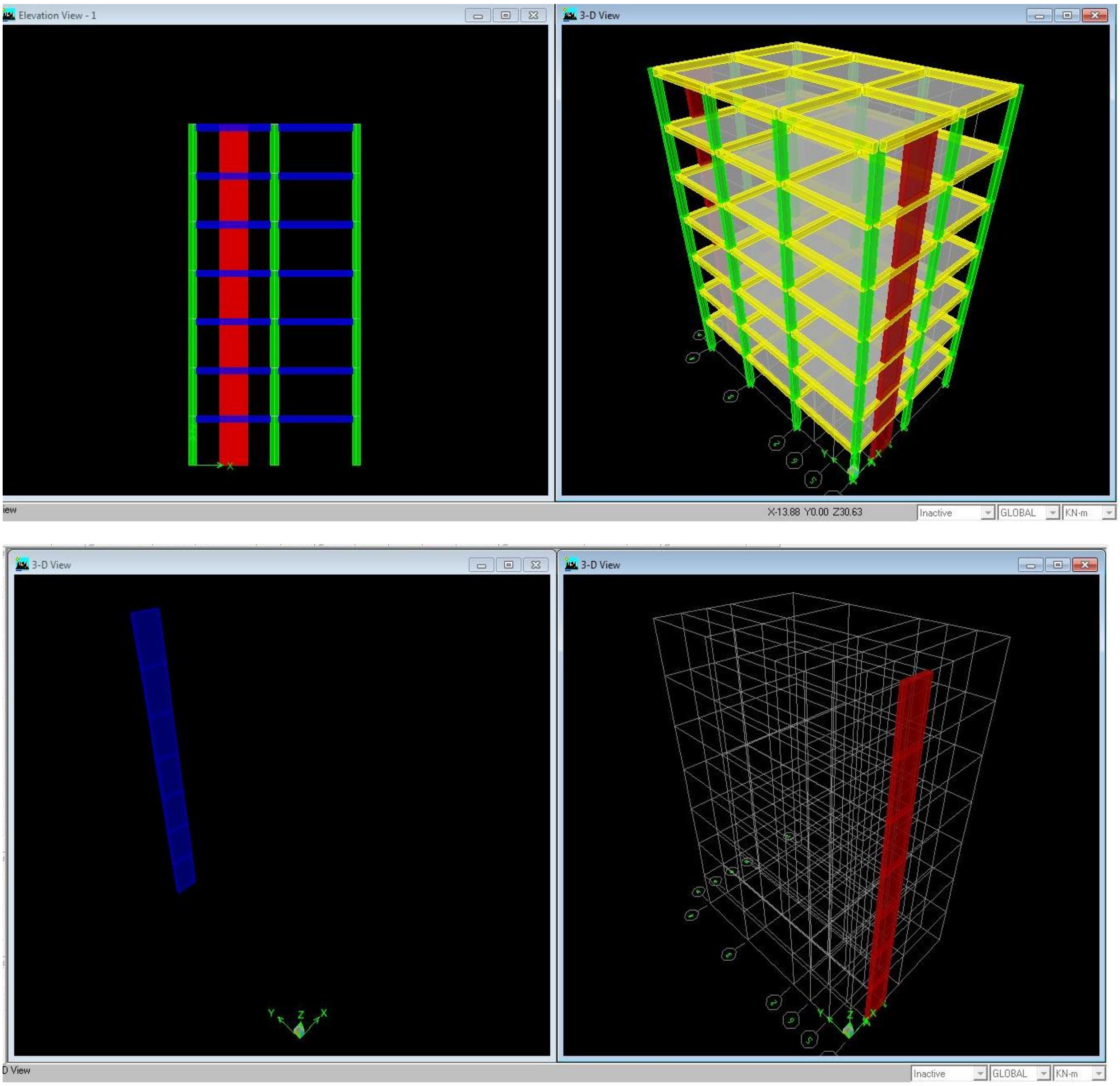
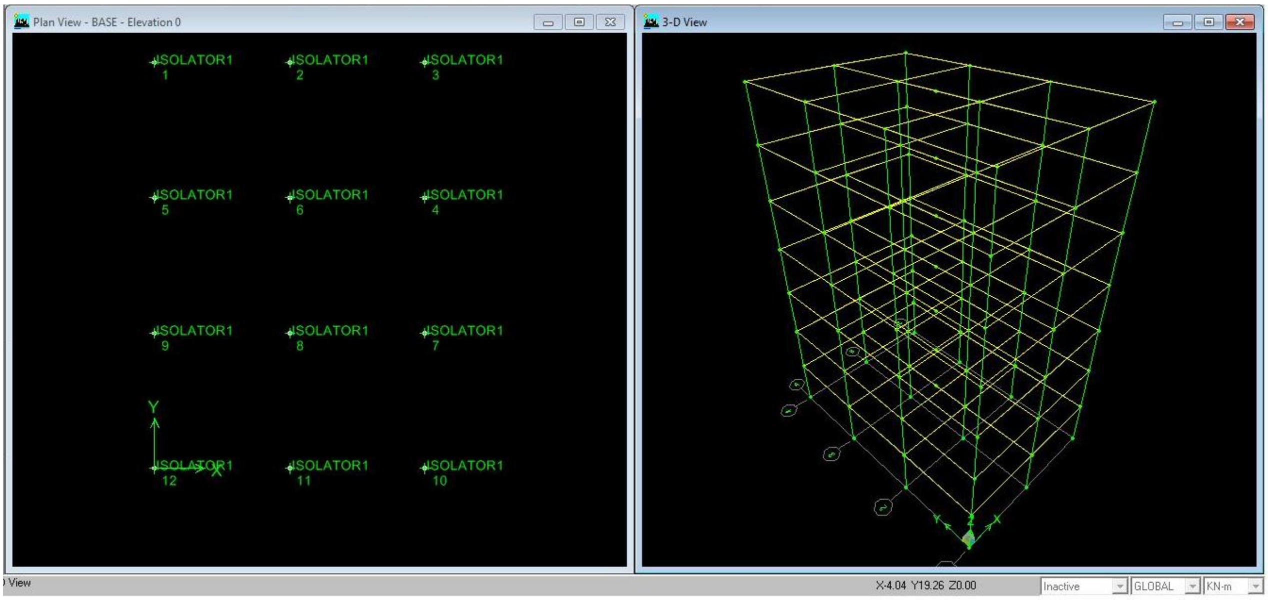


Figure No 8.1 Plan & Elevation of Building Retrofitted With Shear wall

8.2CASE(2) RETROFITTED WITH BASE ISOLATORS

Base Isolators 700mm Dia 350 Mm Height Provided Under Each Columns



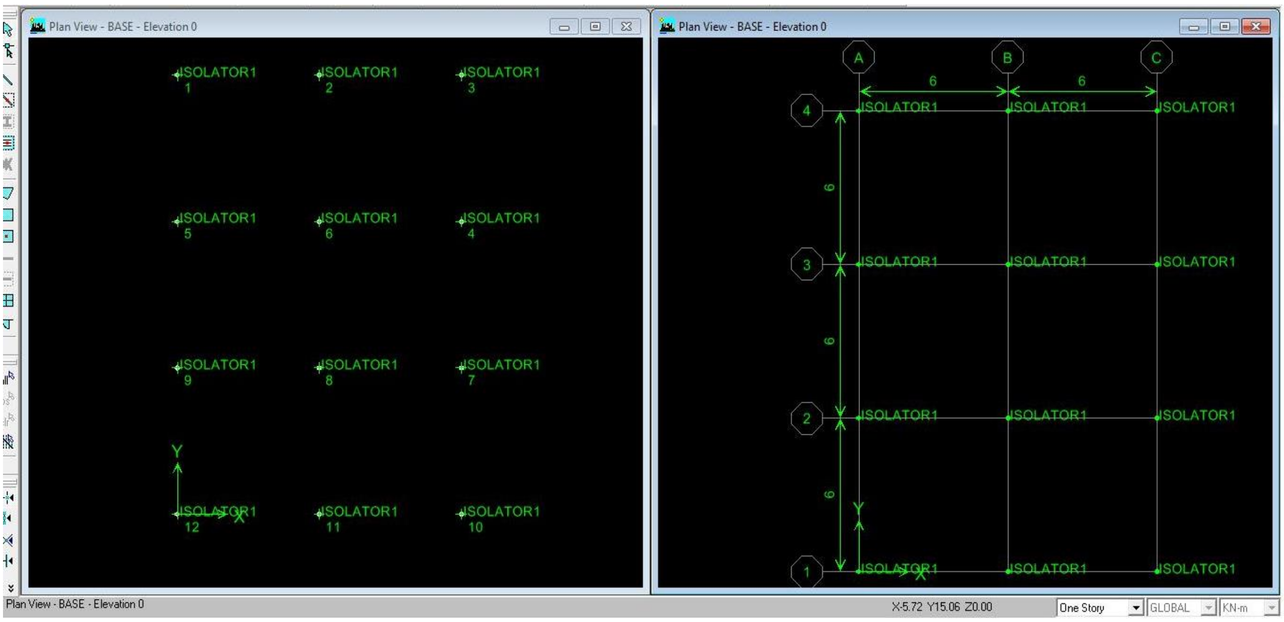


Figure No 8.2:- Plan & Elevation of Building   Retrofitted With Isolators

8.3 CASE 3:- RETROFITTED WITH STEEL BRACINGS

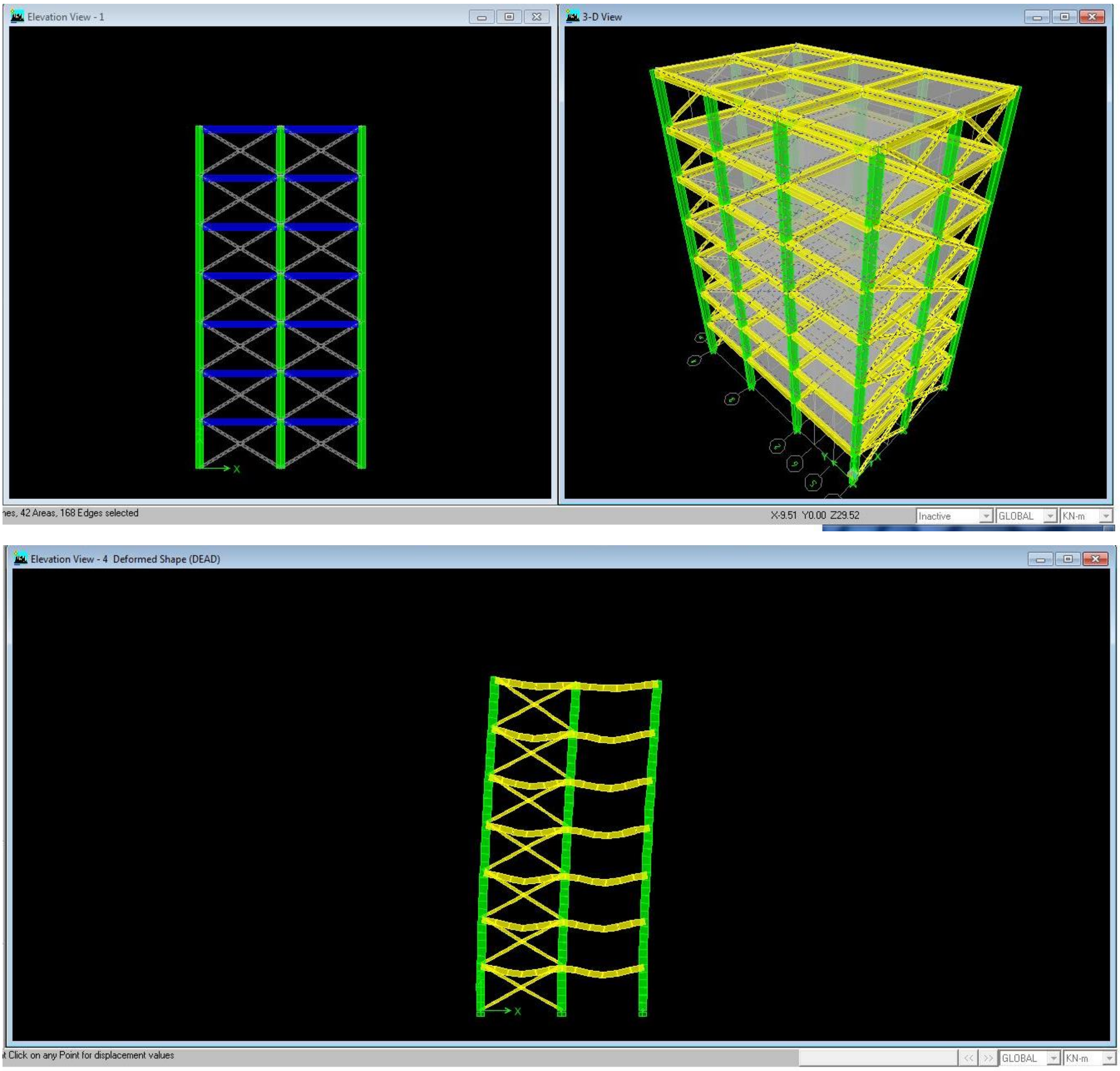


Figure No 8.3 Plan & Elevation of Building Retrofitted With Steel Bracing

Findings and limitations

Single panel bracing good enough to control permissible deflection but element of bracing fail for similar size.

8.4 Results- Graphs& Tables

(A) STOREY DISPLACEMRNT FOR CASE-1, CASE-2, CASE -3

(B) STOREY SHEAR FOR CASE-1, CASE-2,CASE-3

(C) DIAPHRAGM DISPLACEMENT FOR CASE-1, CASE-2,CASE-3

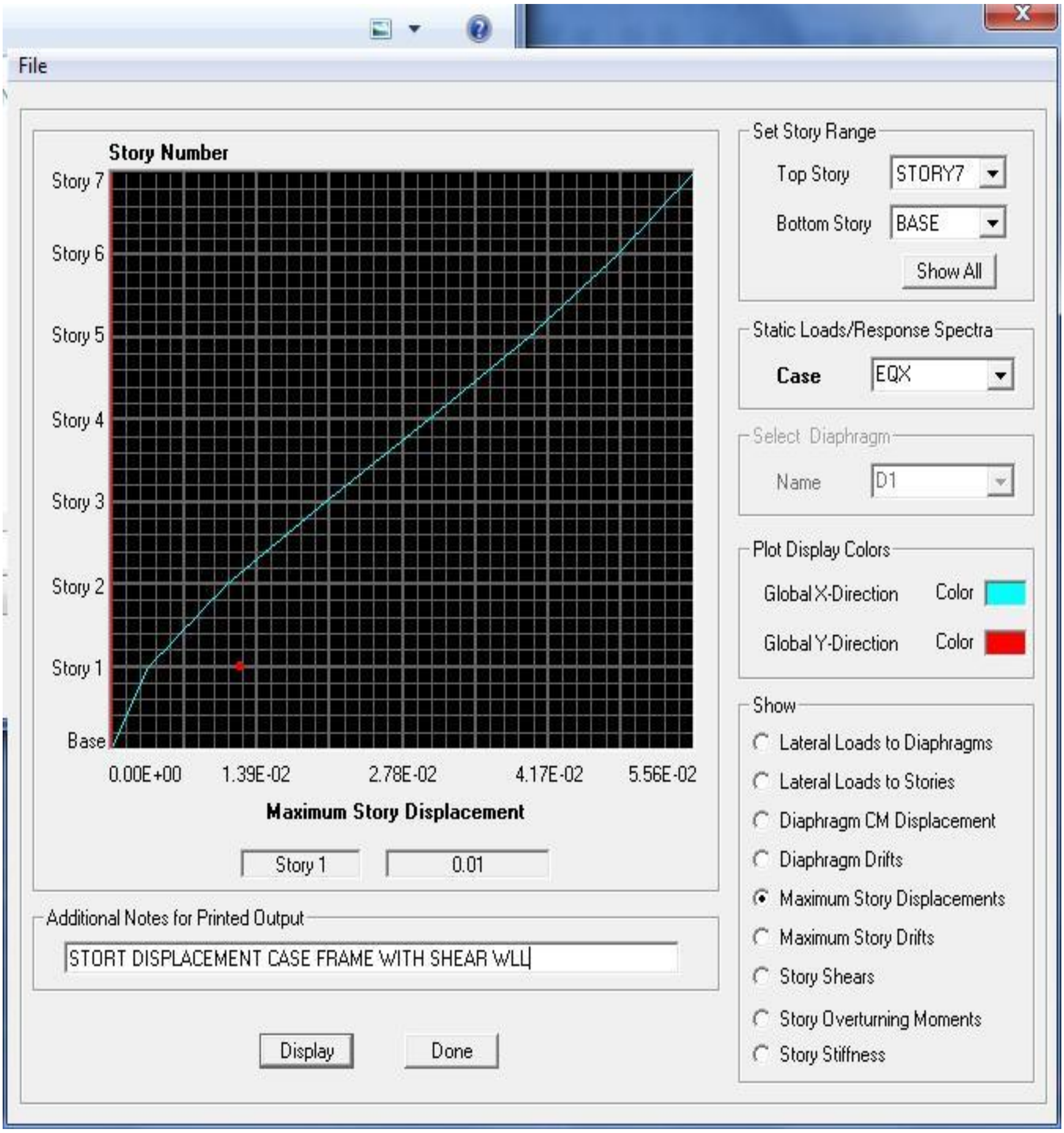


Figure No 8.4 Storey Displacement of Frame with Shear Wall

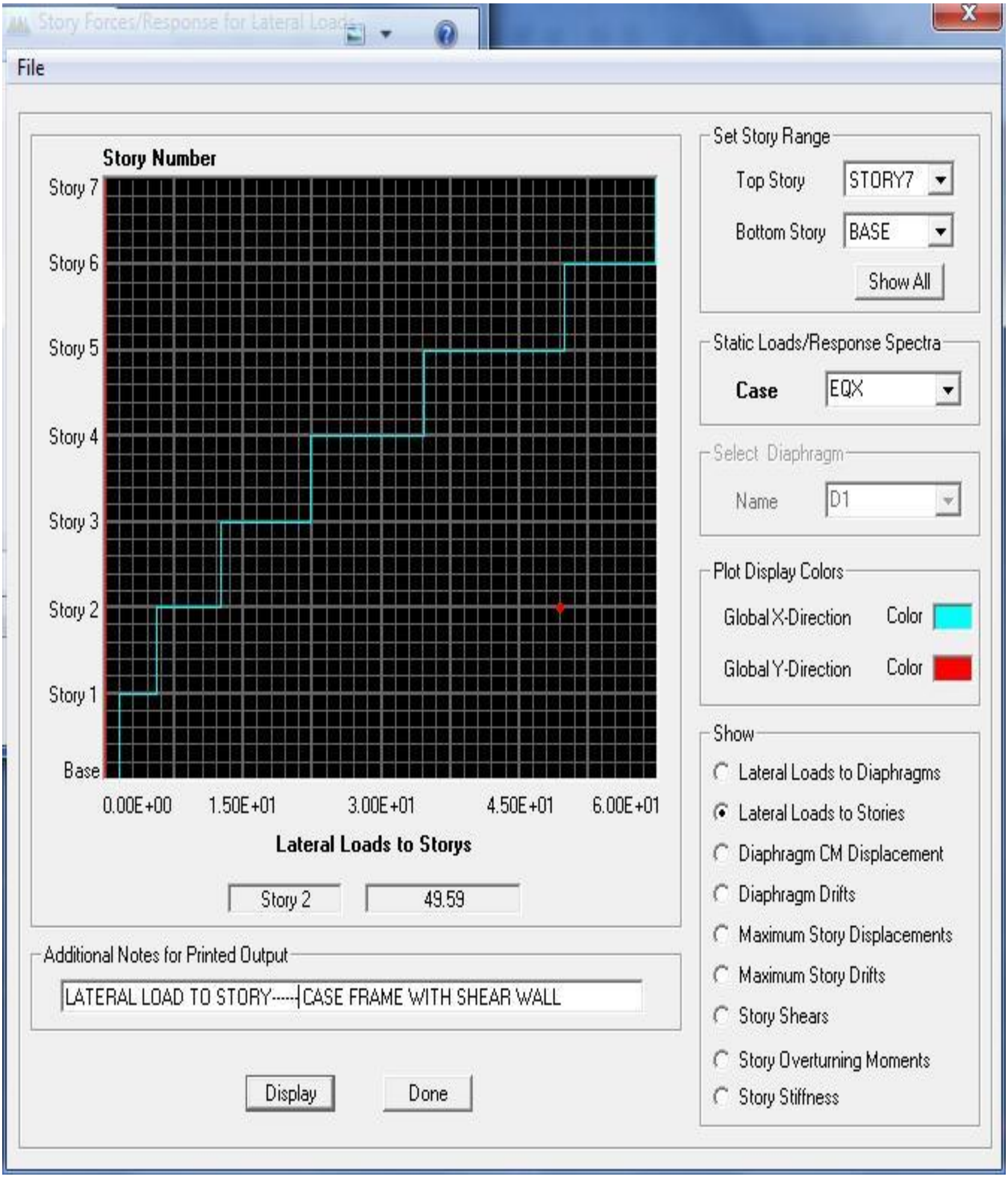


Figure No 8.5 Lateral Load of Frame with Shear Wall

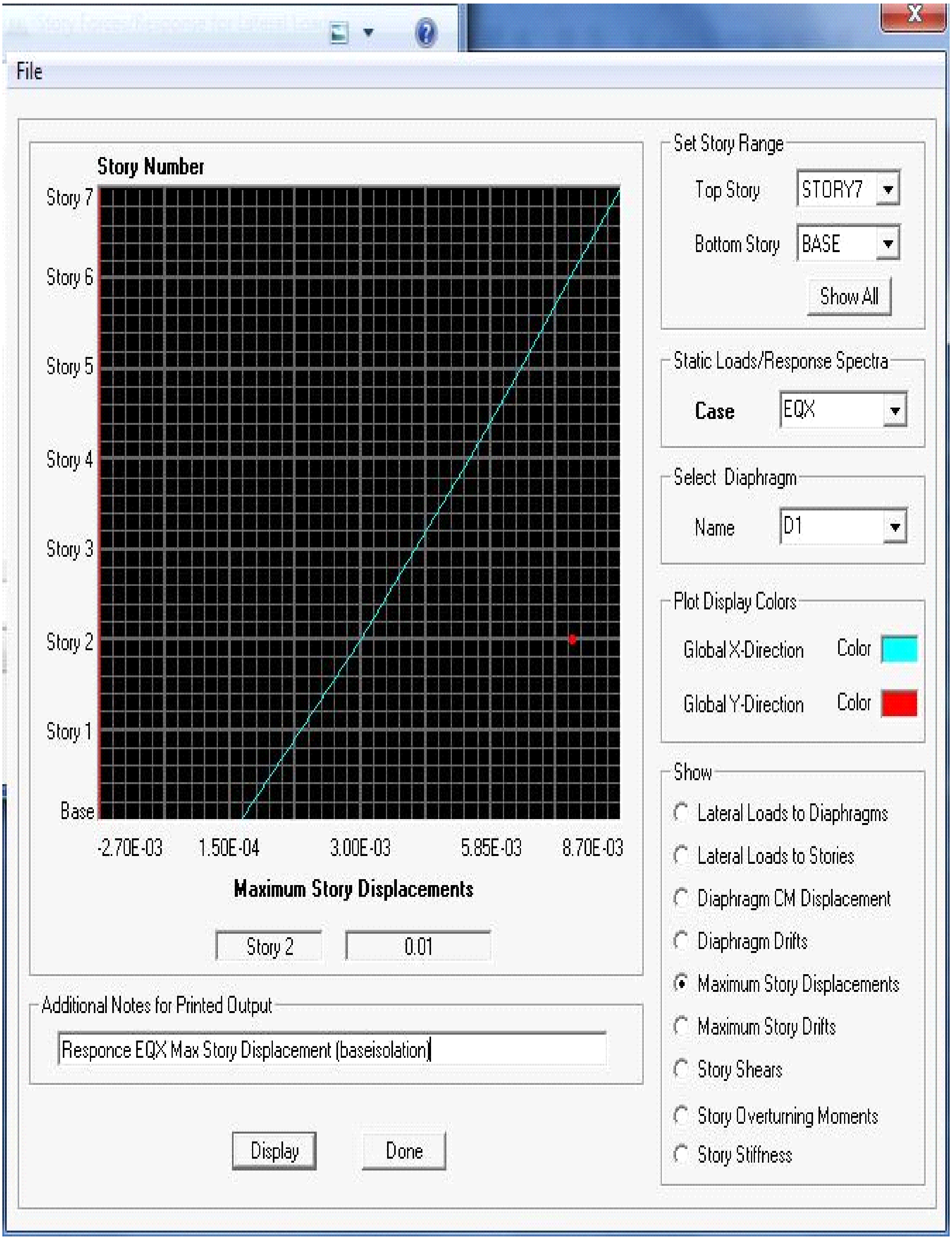


Figure No 8.6 Storey Displacement of frame With Isolation

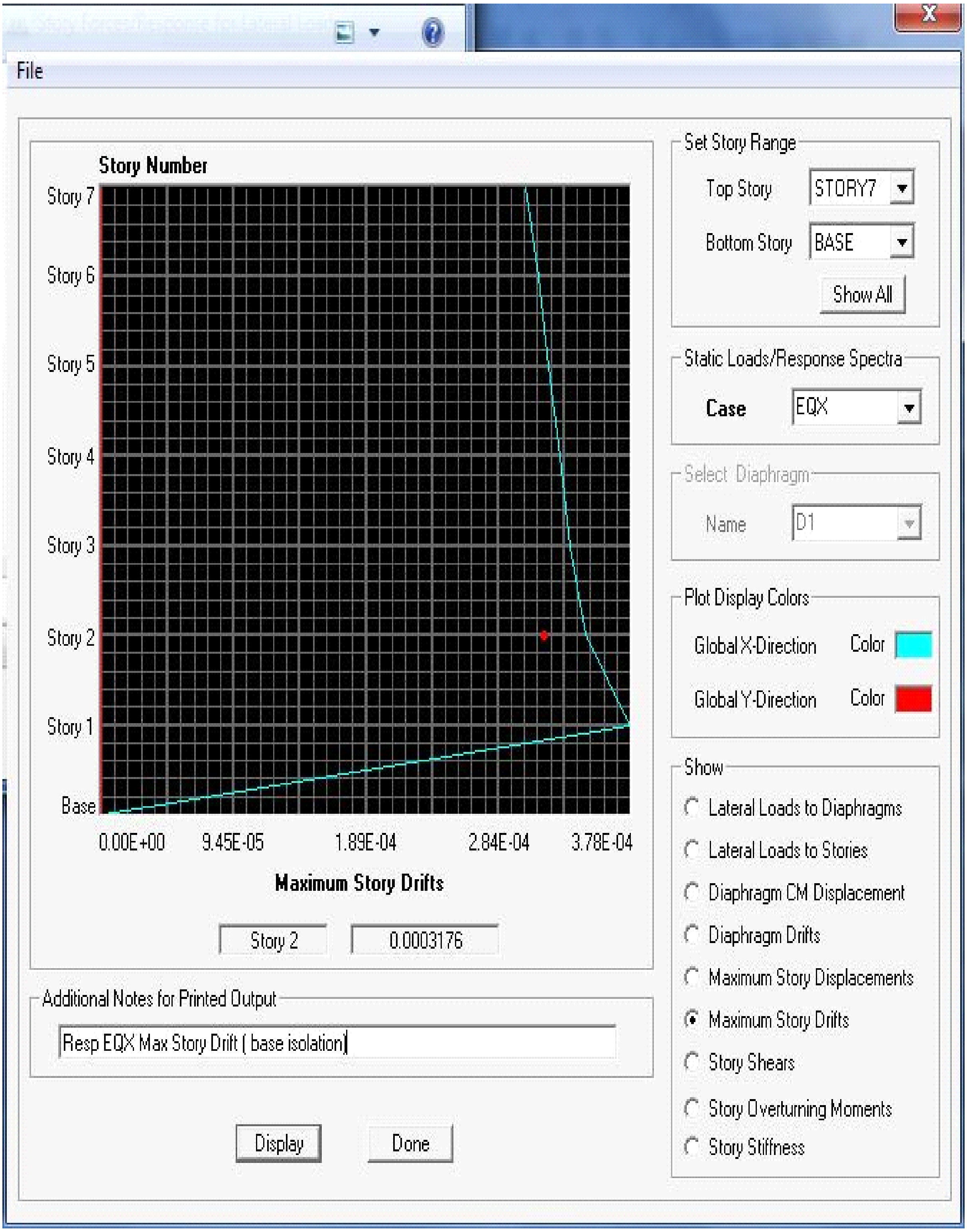


Figure NO 8.7 Storey Drift of frame With Isolation

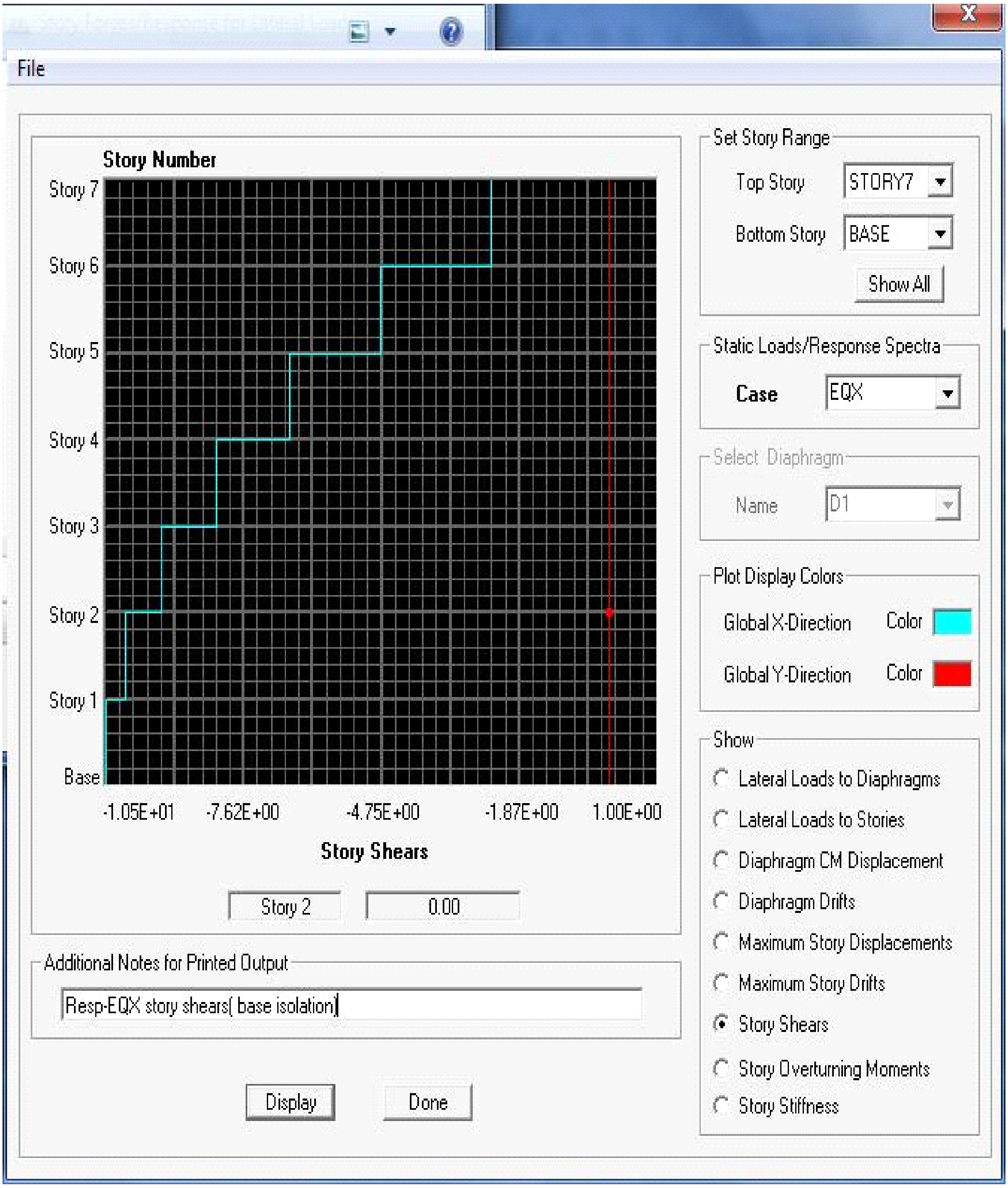


Figure No 8.9 Storey Shear of Frame with Isolation

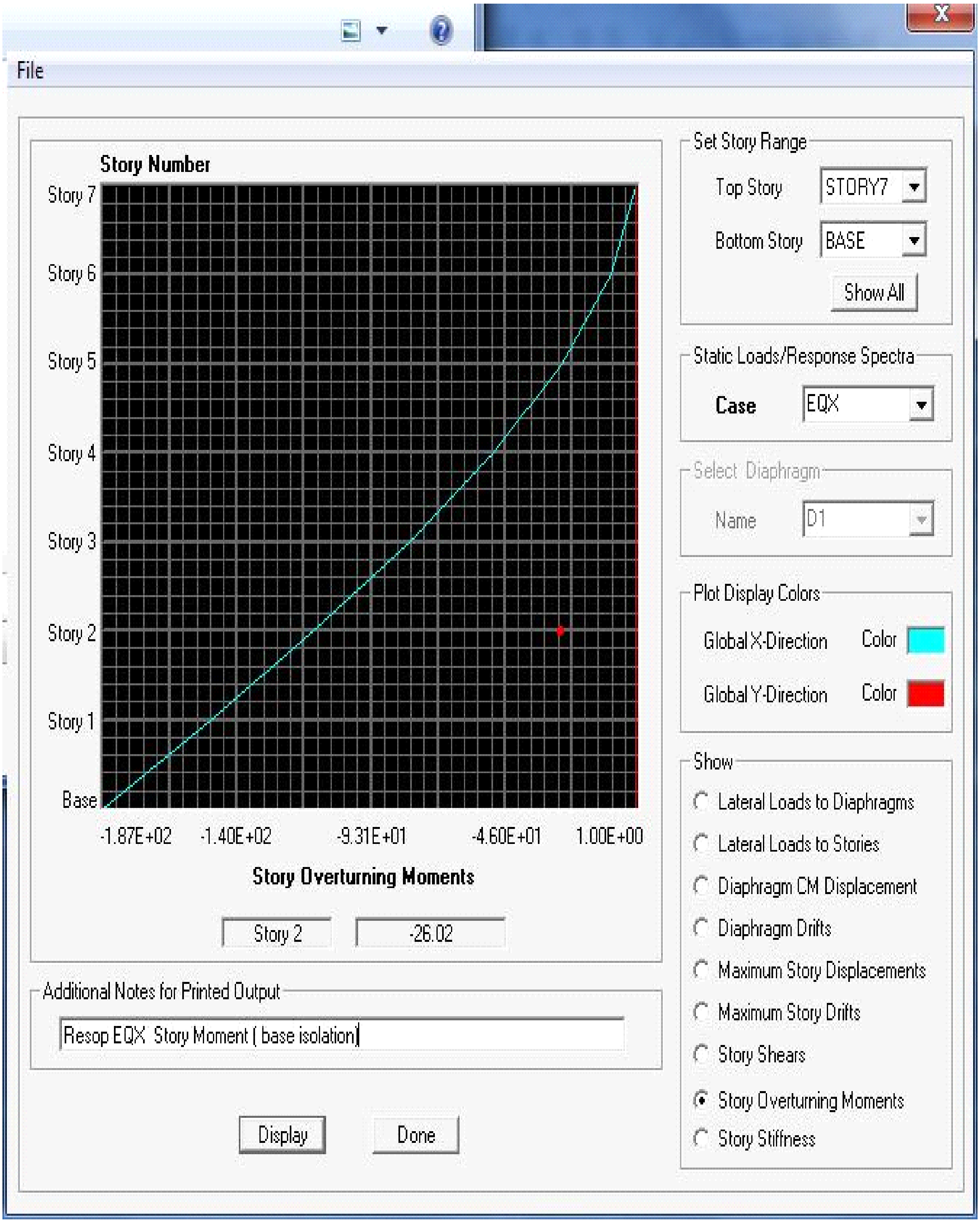


Figure No 8.10 Storey Moment of Frame with Isolation

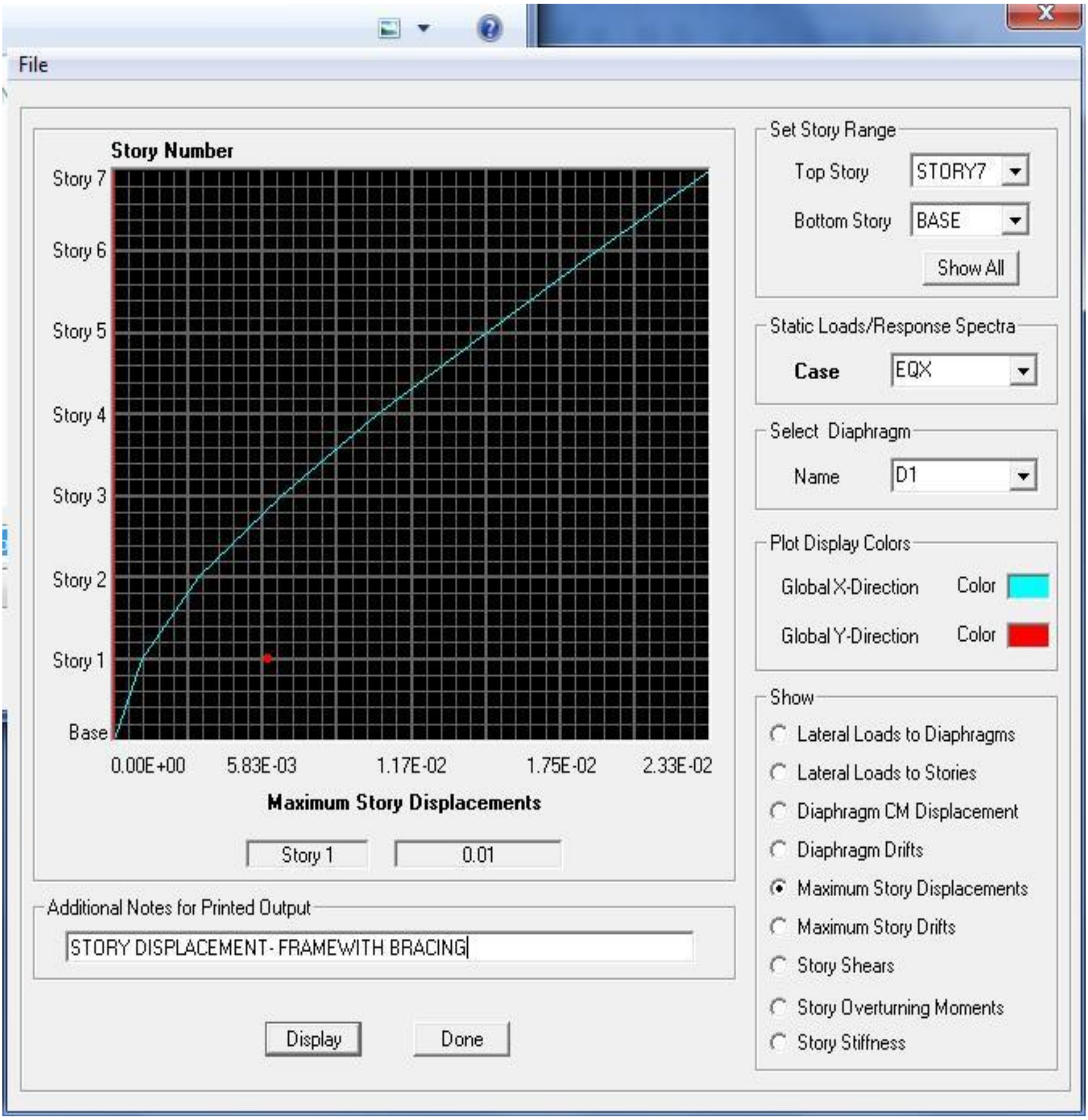


Figure No 8.11 Storey Displacement of Frame With Bracing

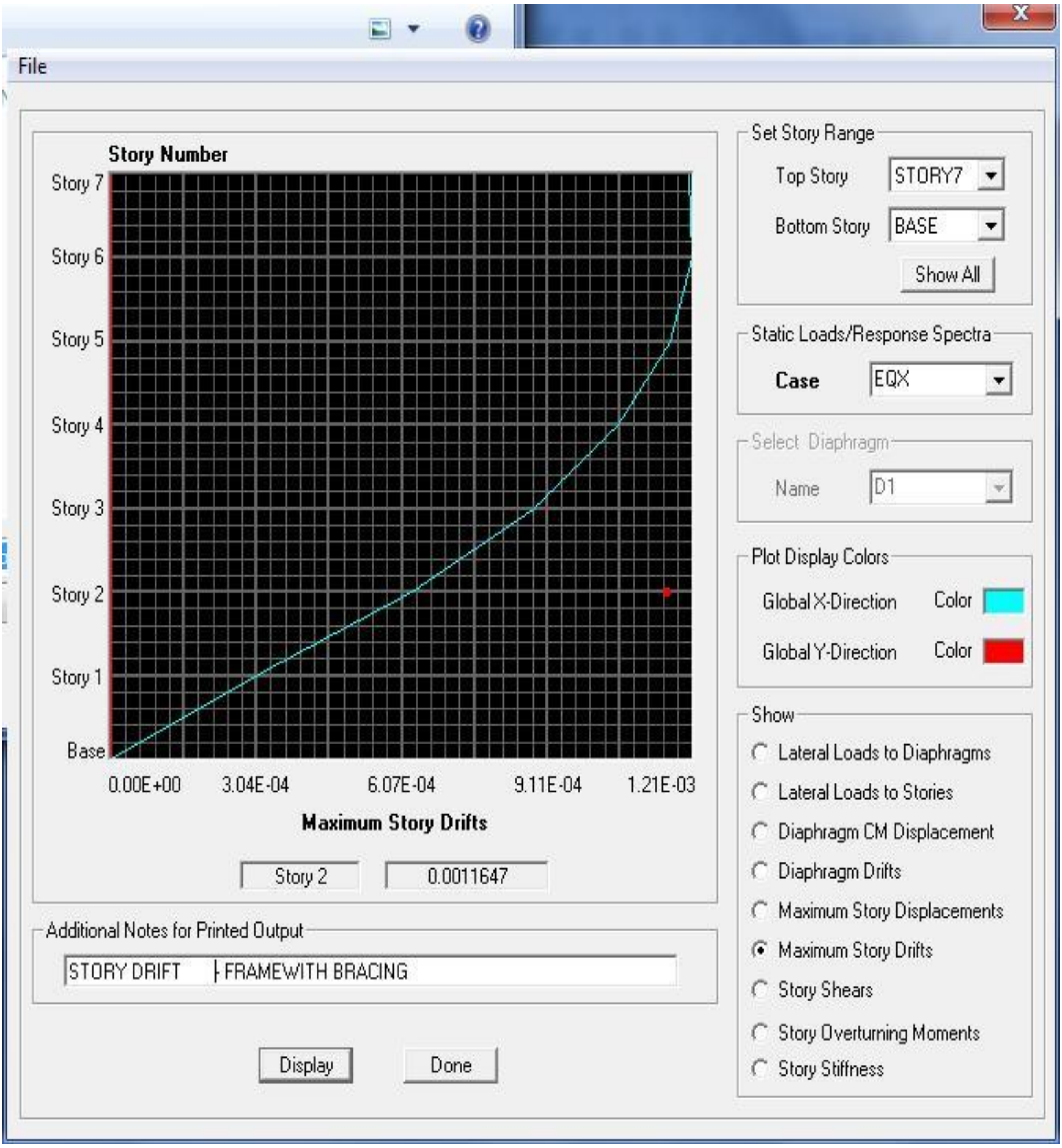


Figure No 8.12 Maximum Storey Drift with Bracing

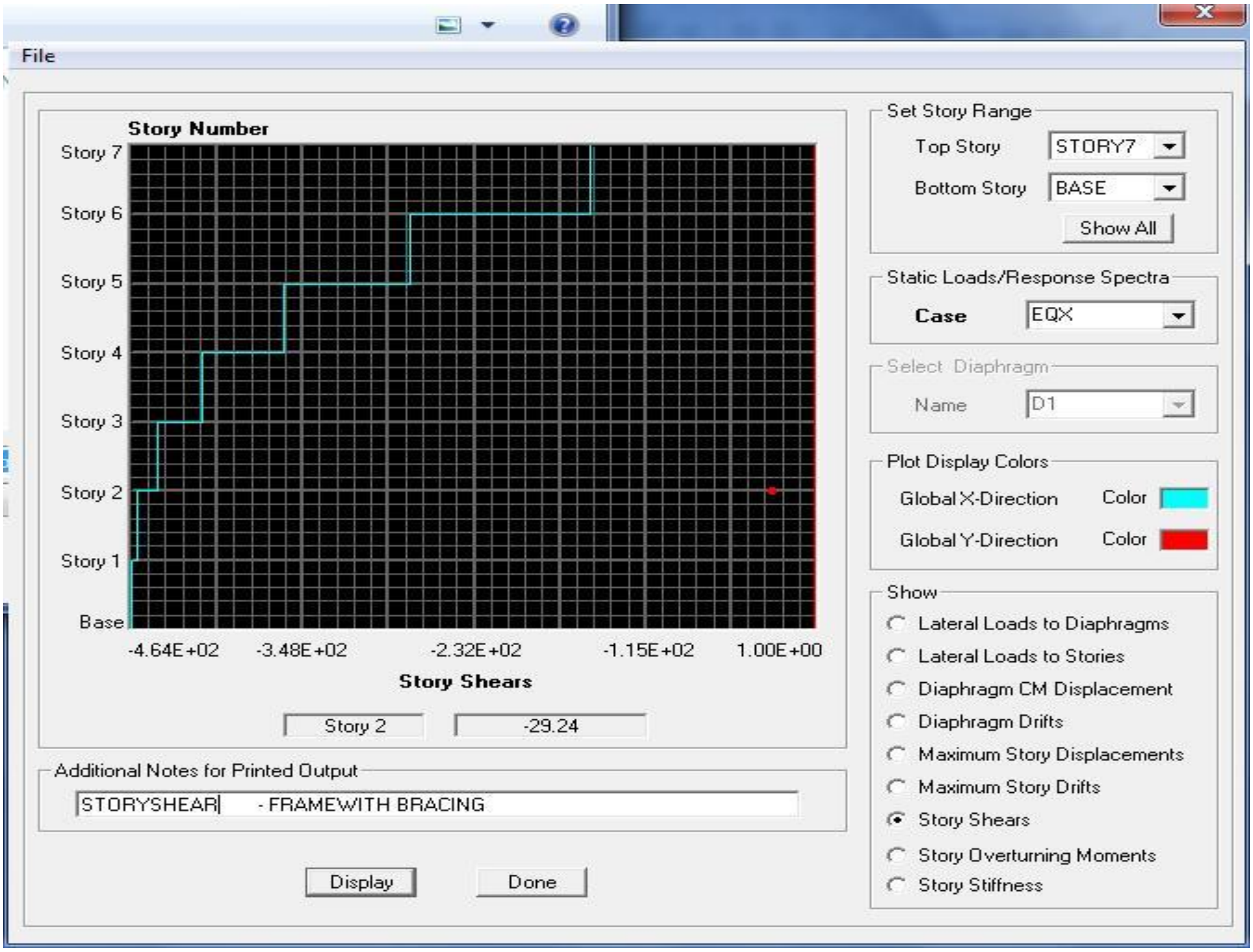


Figure No 8.13 Storey Shear of Frame with Bracing

8.5 RESULT FOR CASE(1)

CASE (1/1) FRAME WITH SHEAR WALL

Table No 8.1:- Diaphragm Displacement With Shear Wall

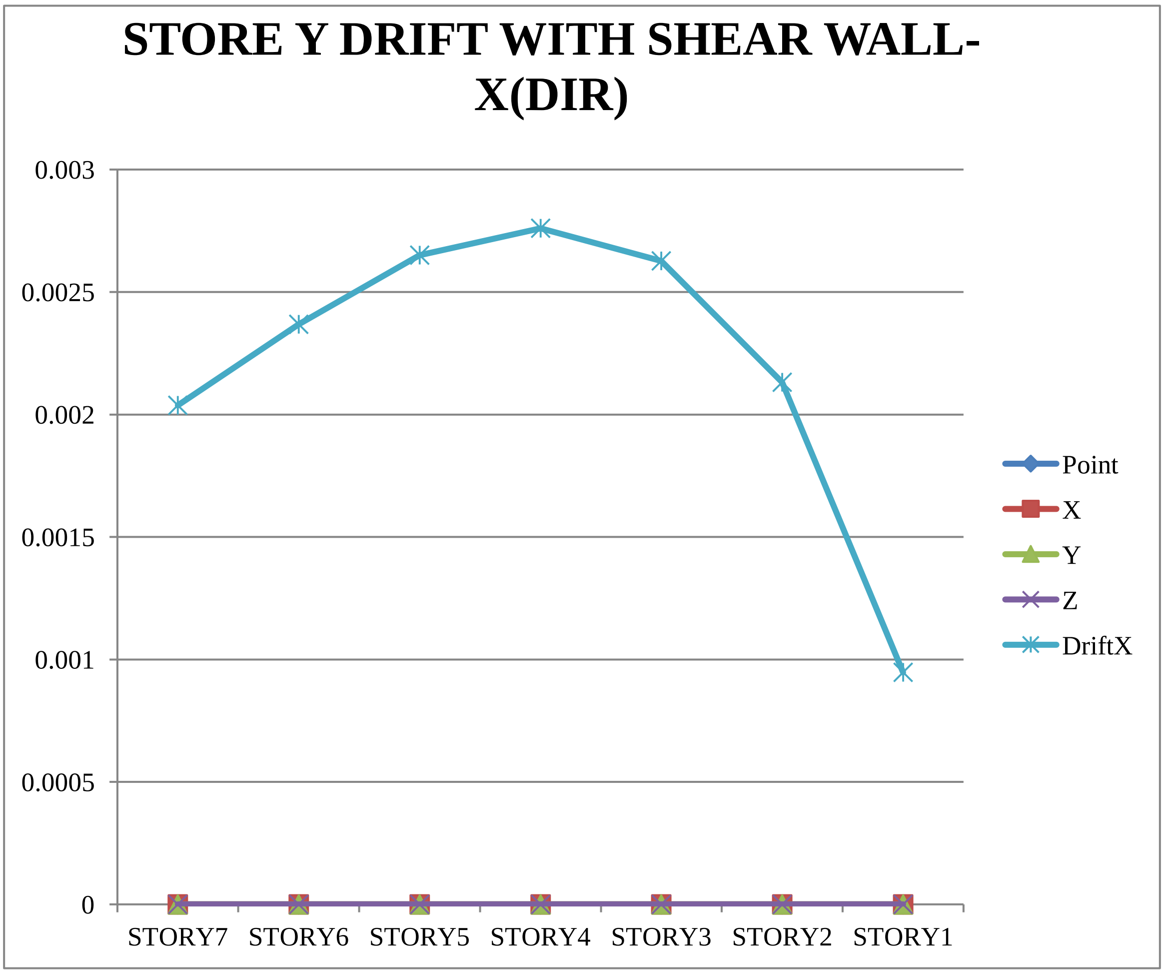
Storey	Diaphragm	Load	UX
STOREY7	D1	EQX	55.8721
STOREY6	D1	EQX	48.5401
STOREY5	D1	EQX	40.0151
STOREY4	D1	EQX	30.4715
STOREY3	D1	EQX	20.5367
STOREY2	D1	EQX	11.081
STOREY1	D1	EQX	3.4106

CASE (1/2)STOREY DRIFT FRAME WITH SHEAR WALL

Table No 8.2 :- Story Drift of Frame With Shear Wall

Story	Drift X(unit m)
STOREY7	0.002037
STOREY6	0.002368
STOREY5	0.002651
STOREY4	0.00276
STOREY3	0.002627
STOREY2	0.002131
STOREY1	0.000947

Figure No 8.14

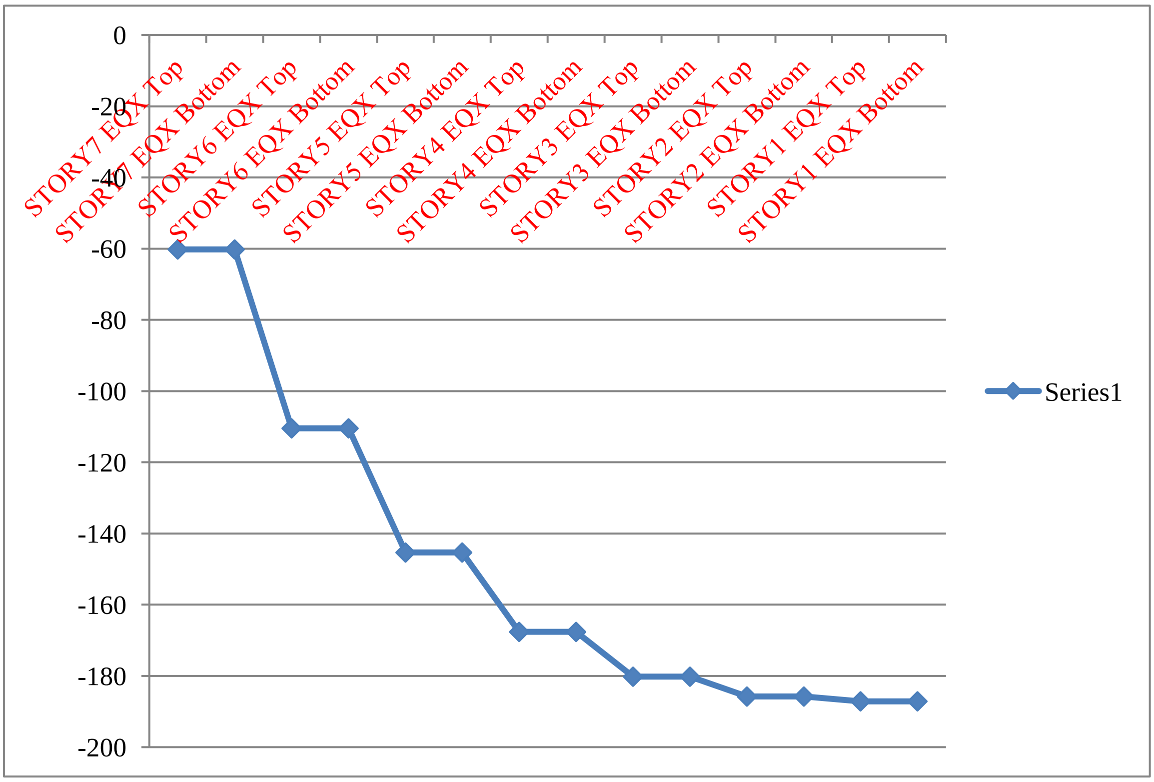


CASE (1/3)STORER SHEAR OF FRAME WITH SHEAR WALL

Table No8.3:- Storey Shear of Frame With Shear Wall

Storey	Load	Loc	VX(KN)
STOREY7	EQX	Top	-60.21
STOREY7	EQX	Bottom	-60.21
STOREY6	EQX	Top	-110.42
STOREY6	EQX	Bottom	-110.42
STOREY5	EQX	Top	-145.29
STOREY5	EQX	Bottom	-145.29
STOREY4	EQX	Top	-167.61
STOREY4	EQX	Bottom	-167.61
STOREY3	EQX	Top	-180.16
STOREY3	EQX	Bottom	-180.16
STOREY2	EQX	Top	-185.74
STOREY2	EQX	Bottom	-185.74
STOREY1	EQX	Top	-187.14
STOREY1	EQX	Bottom	-187.14

Figure No 8.15



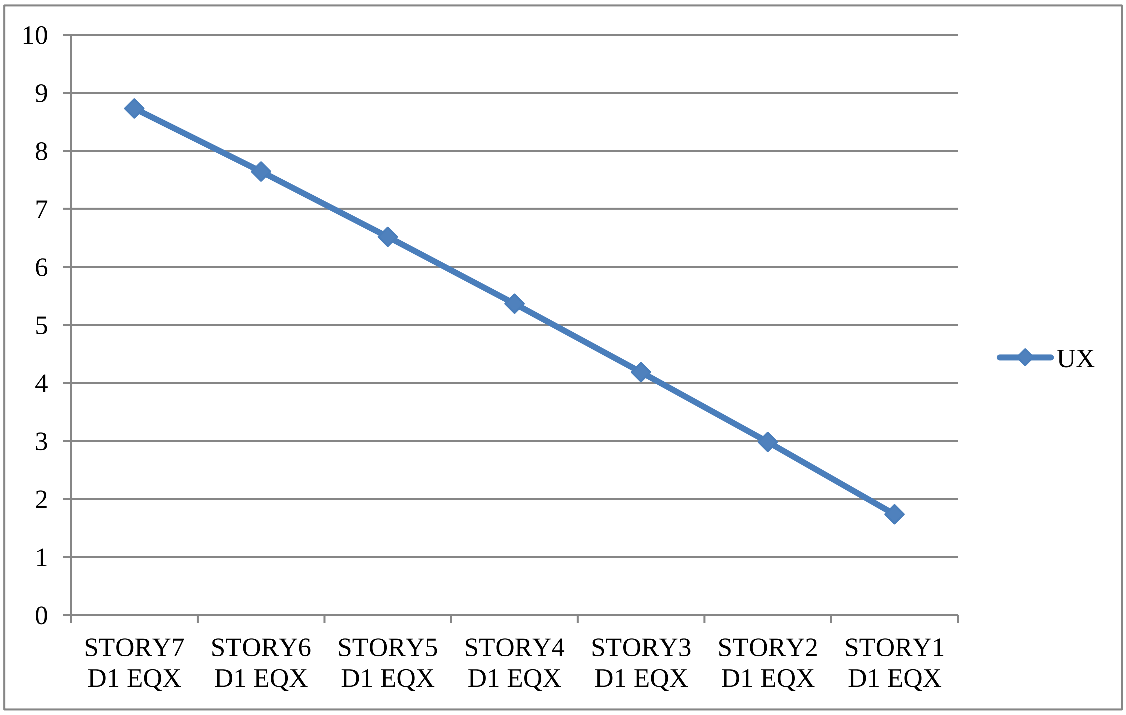
8.6 RESULT FOR CASE(2)

CASE (2/1) FRAME WITH ISOLATOR

Table No 8.4 Diaphragm Displacement With Isolators

Storey	Diaphragm	Load	UX(mm)
STOREY7	D1	EQX	8.7301
STOREY6	D1	EQX	7.6403
STOREY5	D1	EQX	6.5188
STOREY4	D1	EQX	5.366
STOREY3	D1	EQX	4.1859
STOREY2	D1	EQX	2.9797
STOREY1	D1	EQX	1.7341

Figure No 8.16 Diaphragm Displacement With Isolators



CASE (2/2) STOREY DRIFT OF FRAME WITH ISOLATOR

Table No 8.5:- Storey Drift of Frame With Isolator

STOREY DRIFT IN mm(frame + isolators)Unit m Direction =X	
Storey	Drift X
STOREY7	0.000303
STOREY6	0.000312
STOREY5	0.00032
STOREY4	0.000328
STOREY3	0.000335
STOREY2	0.000346
STOREY1	0.000378

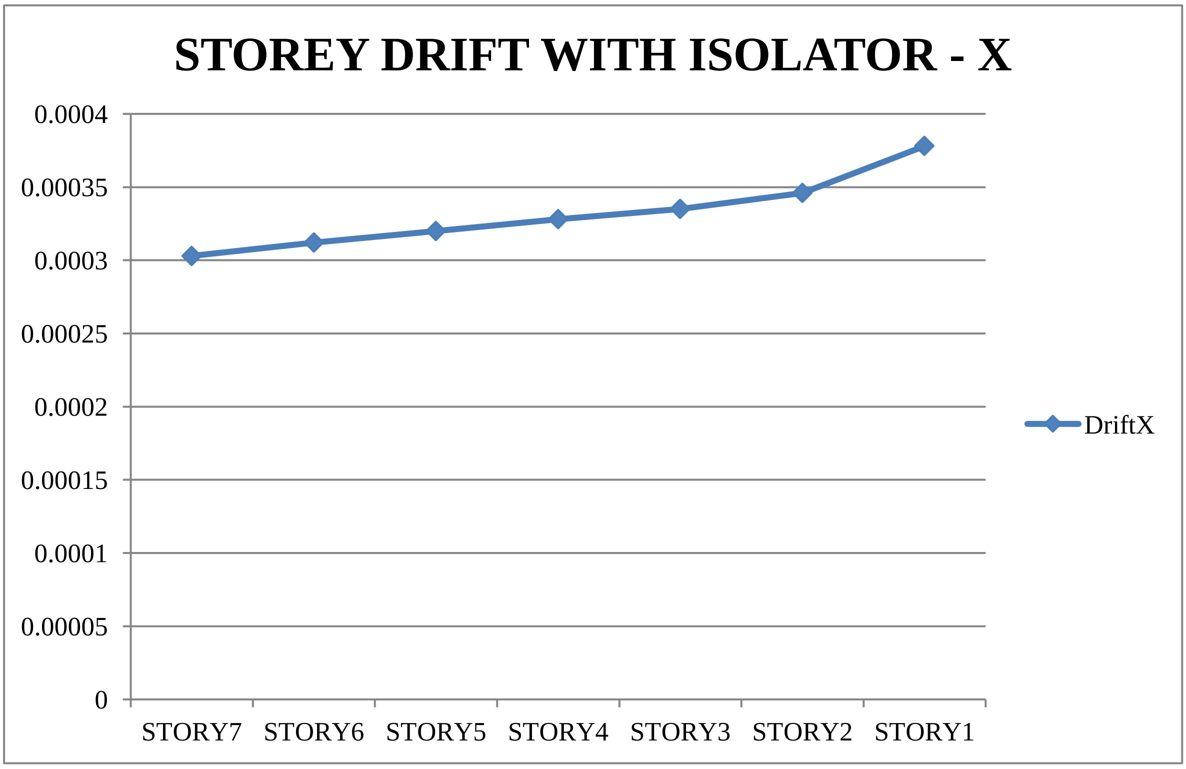


Figure No 8.17 Storey Drift of Frame With Isolator

CASE (2/3) STOREY SHEAR OF FRAME WITH ISOLATOR

Table No8.6:- Storey Shear of Frame With Isolator

Unit KN/M in x direction

Storey	Load	Loc	VX
STOREY7	EQX	Top	-2.45
STOREY7	EQX	Bottom	-2.45
STOREY6	EQX	Top	-4.75
STOREY6	EQX	Bottom	-4.75
STOREY5	EQX	Top	-6.66
STOREY5	EQX	Bottom	-6.66
STOREY4	EQX	Top	-8.19
STOREY4	EQX	Bottom	-8.19
STOREY3	EQX	Top	-9.34
STOREY3	EQX	Bottom	-9.34
STOREY2	EQX	Top	-10.1
STOREY2	EQX	Bottom	-10.1
STOREY1	EQX	Top	-10.49
STOREY1	EQX	Bottom	-10.49

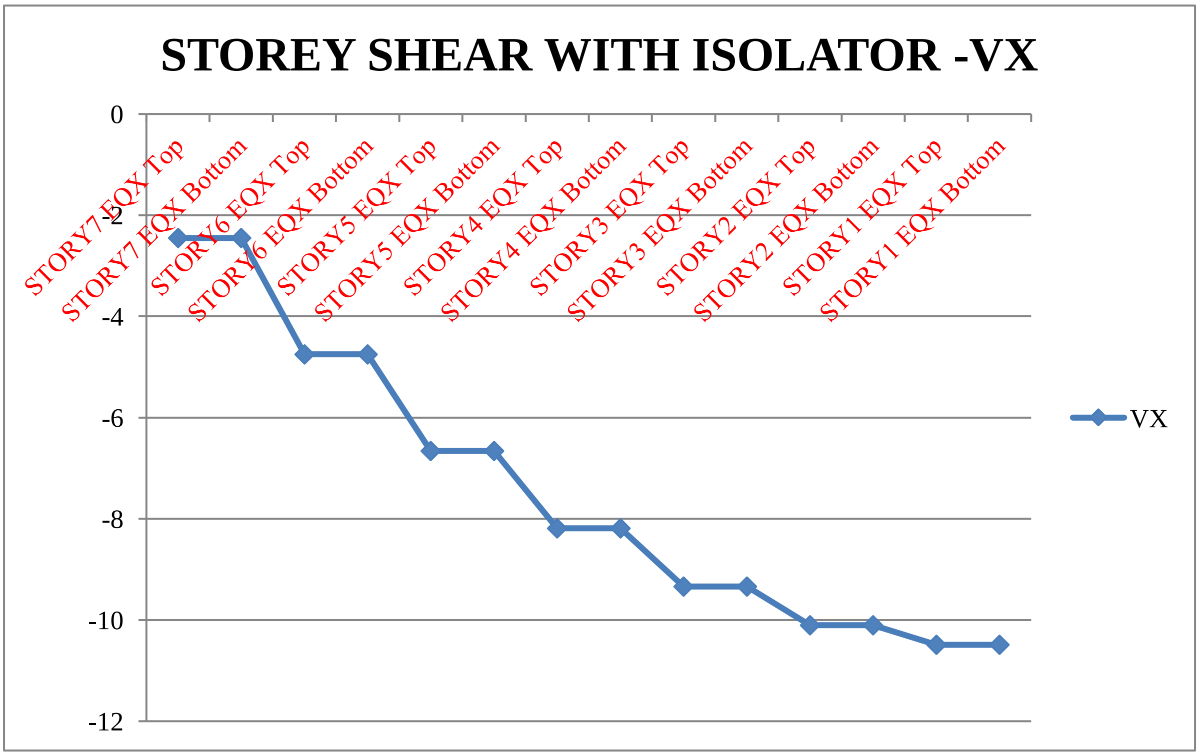


Figure No 8.18 storey shear of frame with Isolator

8.7 RESULT FOR CASE(3)

case (3/1) DIAPHRAGM DISPLACEMENT IN X DIRECTION BRACING

Table No 8.7:- Diaphragm Displacement With Bracing

Storey	Diaphragm	Load	UX(mm)
STOREY7	D1	EQX	23.2613
STOREY6	D1	EQX	18.9132
STOREY5	D1	EQX	14.5434
STOREY4	D1	EQX	10.3361
STOREY3	D1	EQX	6.5162
STOREY2	D1	EQX	3.3389
STOREY1	D1	EQX	1.073

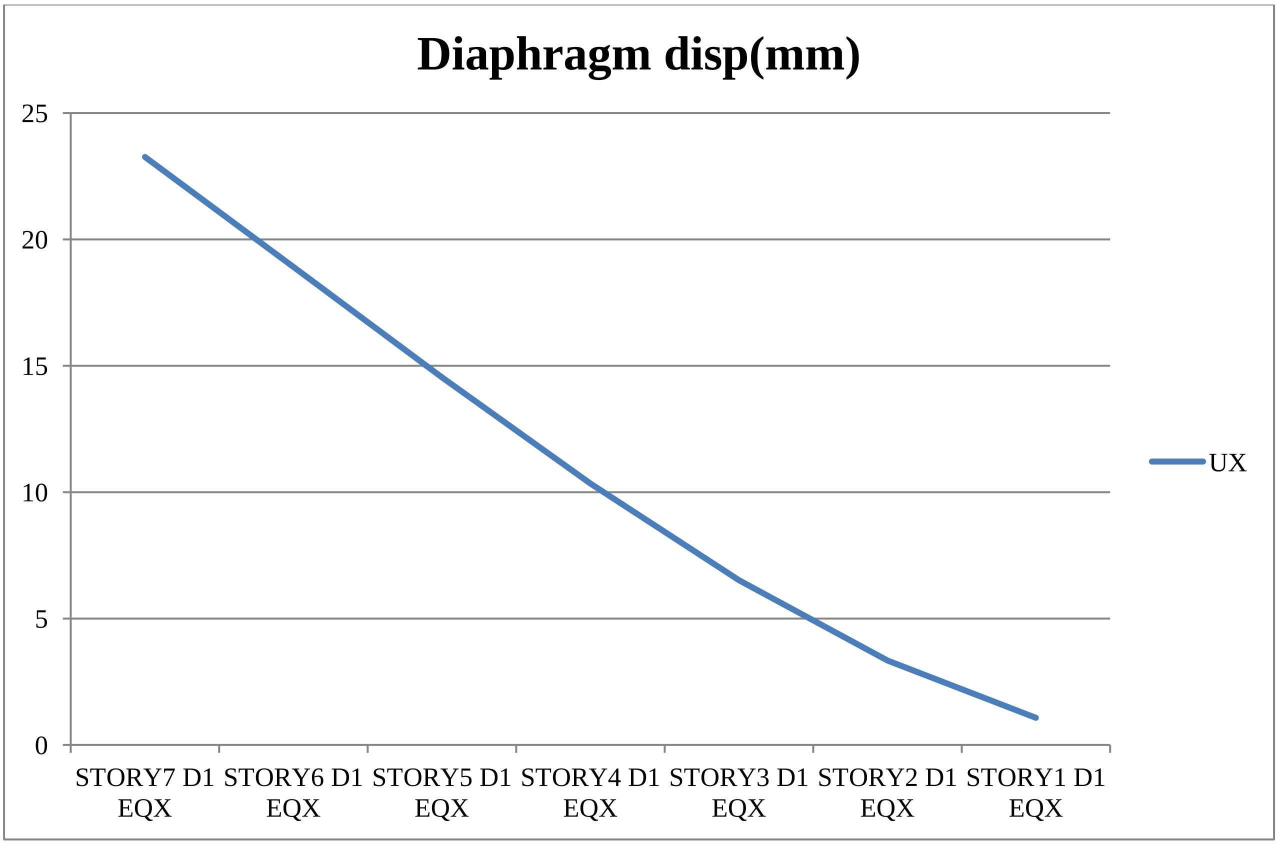


Figure No 8.19 Diaphragm Displacement With Bracing

CASE(3/2) STOREY DRIFT WITH BRACING

Table No 8.8:- Storey Drift of Frame With Bracing

Story	Drift X(Deflection in X Drection)
STOREY7	0.001208
STOREY6	0.001214
STOREY5	0.001169
STOREY4	0.001061
STOREY3	0.000883
STOREY2	0.000629
STOREY1	0.000298

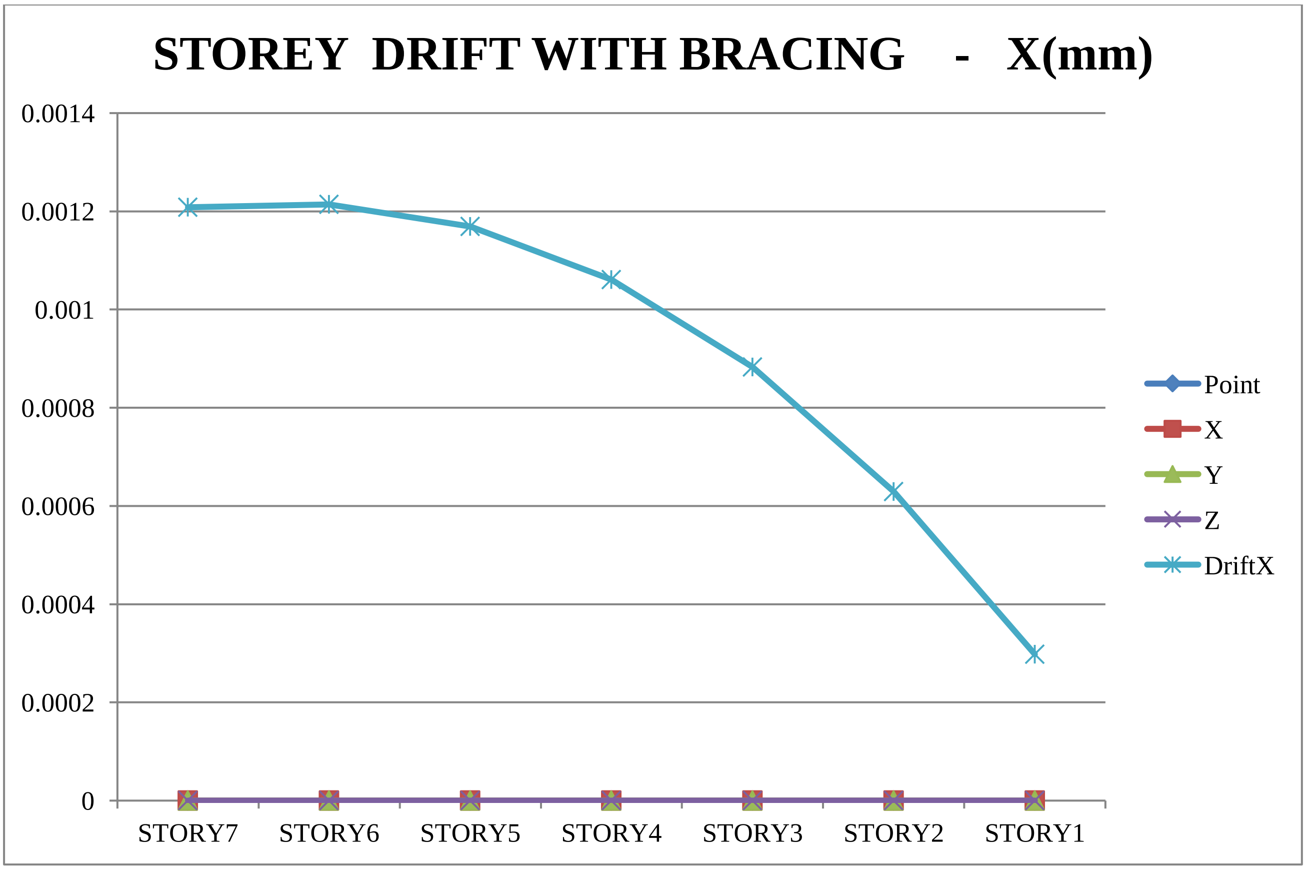


Figure No8.20 Storey Drift of Frame With Bracing

CASE (3/3) STOREY SHEAR WITH BRACING

Table No8.9:- Storey Shear of Frame With Bracing

Storey	Load	Loc	VX(X Direction)
STOREY7	EQX	Top	VX
STOREY7	EQX	Bottom	-152.17
STOREY6	EQX	Top	-152.17
STOREY6	EQX	Bottom	-275.53
STOREY5	EQX	Top	-275.53
STOREY5	EQX	Bottom	-361.21
STOREY4	EQX	Top	-361.21
STOREY4	EQX	Bottom	-416.04
STOREY3	EQX	Top	-416.04
STOREY3	EQX	Bottom	-446.88
STOREY2	EQX	Top	-446.88
STOREY2	EQX	Bottom	-460.59
STOREY1	EQX	Top	-460.59
STOREY1	EQX	Bottom	-464.01
Base shear		Ground level	464.0KN

Discussion: From comparison , we observe the following points

1) Storey shear is linear in a storey height

2) Value constant at a junction of slab

Chapter 9 COMPARISON AND FINDINGS

9.0 Preamble:-

Comparison have been made for the values obtained from shear wall frame , base Isolated Frame and Braced frame analysis. Values are noted for X-direction only. although values for Y-direction, Z-direction are available with software.

9.1 Comparison For Diaphragm Deflection (X Direction)

Table No:- 9.1 Comparison For Diaphragm Deflection

COMPARATIVE STATEMENT DIAPHRAGM DEFLECTION (mm) U X							
DIAPH	EQ FORCE	STOREY N0.	HEIGHT (M)	Old str	CASE(1)	CASE(2)	CASE(1)
D1	EQX	STOREY6	21.6	24.8967	48.5401	7.6403	18.9132
D1	EQX	STOREY5	18	21.5635	40.0151	6.5188	14.5434
D1	EQX	STOREY4	14.4	17.2799	30.4715	5.366	10.3361
D1	EQX	STOREY3	10.8	12.3809	20.5367	4.1859	6.5162
D1	EQX	STOREY2	7.2	7.2595	11.081	2.9797	3.3389
D1	EQX	STOREY1	3.6	2.5564	3.4106	1.7341	1.073

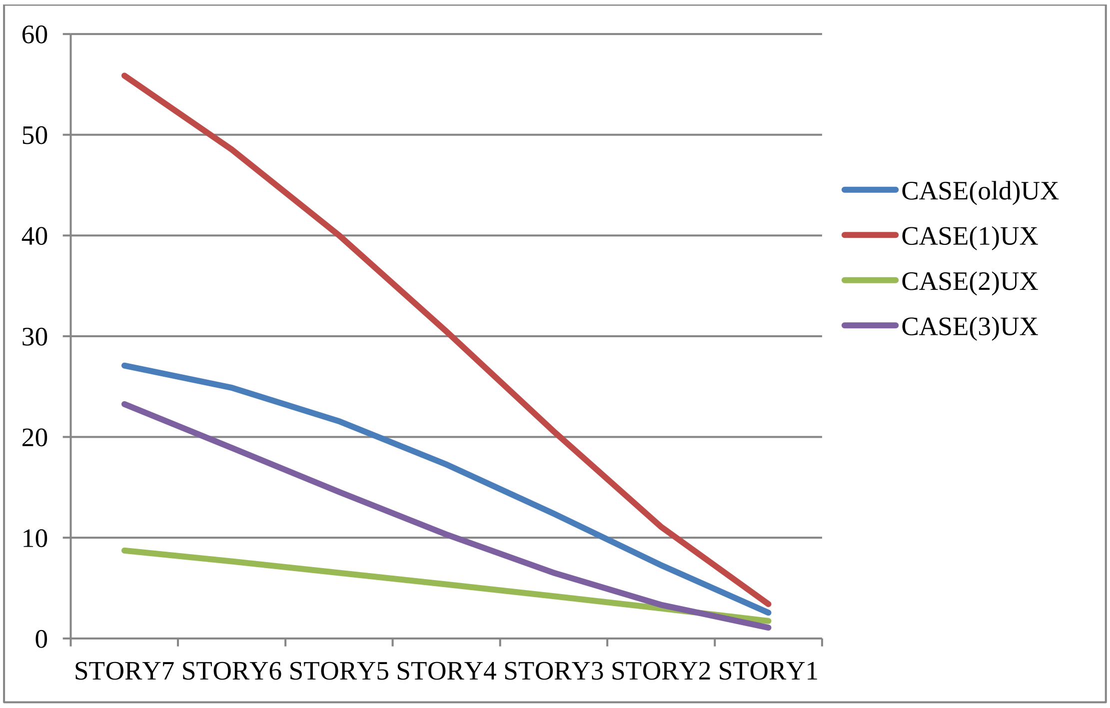


Figure No 9.1 Comparison of Diaphragm Deflection (X Direction)

Discussion: From comparison , we observe the following points

(1) Overall deflection of Base Isolated building much lesser than others.

(2) Bracing has less deflection than the shear wall.

9.2 Comparison For Storey Deflection

Table No.9.2:- Comparison For Storey Deflection

COMPARATIVE STATEMENT STORY DEFLECTOIN (Drift x m) U X					
STOREY NO.	DIAPH.No.	CASE(old)UX	CASE(1)UX	CASE(2)UX	CASE(3)UX
STOREY7	D1	0.00061	0.002037	0.000303	0.001694
STOREY6	D1	0.000926	0.002368	0.000312	0.002242
STOREY5	D1	0.00119	0.002651	0.00032	0.002681
STOREY4	D1	0.001361	0.00276	0.000328	0.002917
STOREY3	D1	0.001423	0.002627	0.000335	0.002932
STOREY2	D1	0.001306	0.002131	0.000346	0.002678
STOREY1	D1	0.00071	0.000947	0.000378	0.001334

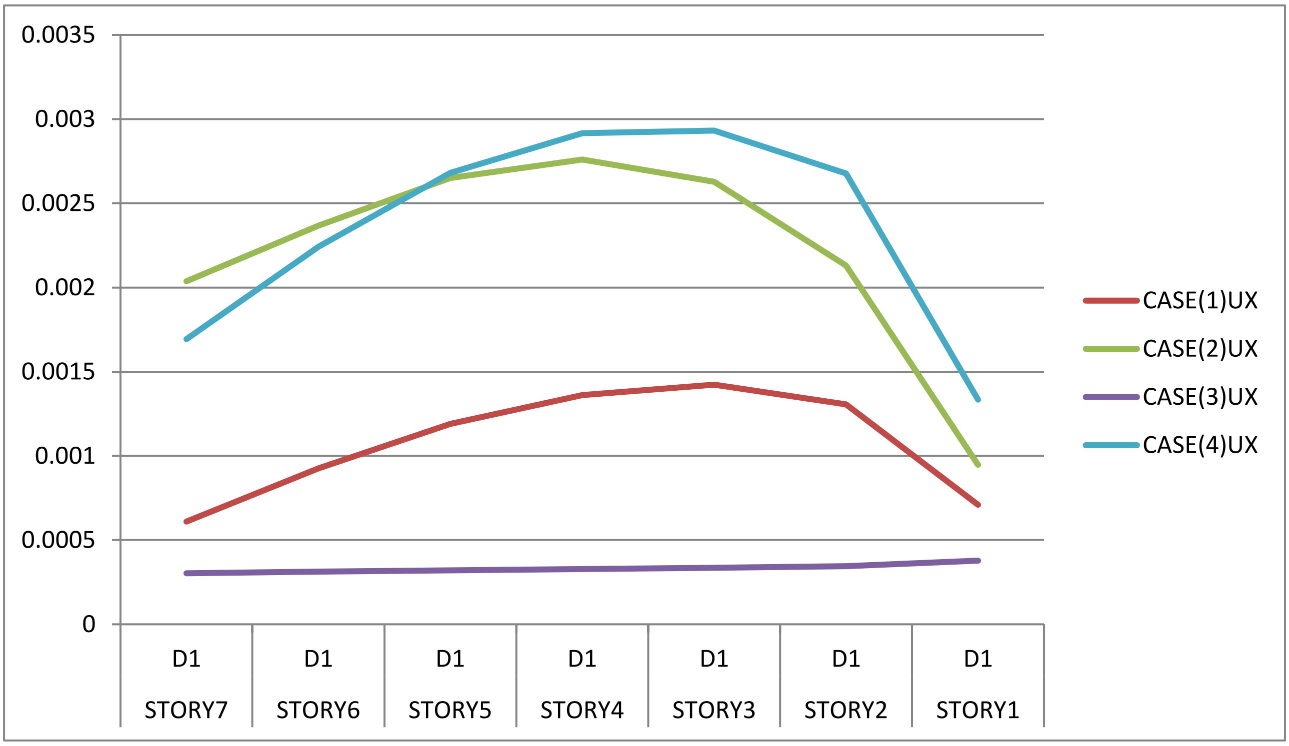


Figure No 9.2 Comparison of Storey Deflection (X Direction)

Discussion: From comparison , we observe the following points

I) As per IS1893(Part1) :2002- The storey drift in any storey due to the lateral force hall not exceed 0.004 times the storey height(limiting Deflection value=H/400).The maximum deflection limit in this case=21.6X0.004=0.086M(86.4mm)

II) Cross Bracing to be placed with in full panel (6.0mX6.0m).

III) Story deflection for base isolated building lesser than others.

IV) Bracing is better than shear wall for story deflection consideration

Lateral deflection

(a) Base isolation=25%of shear wall

(b) Base isolation=30%of Bracing systems

9.3 :-COMPARISON FOR STORY SHEAR

Table No 9.3 comparison for storey shear

Story	Loc	CASE(old)VX	CASE(1)VX	CASE(2)VX	CASE(3)VX	height
STORY7	Top	-105.95	-60.21	-2.45	-50.55	25.2
STORY7	Bottom	-105.95	-60.21	-2.45	-50.55	21.6
STORY6	Top	-190.91	-110.42	-4.75	-91.18	21.6
STORY6	Bottom	-190.91	-110.42	-4.75	-91.18	18
STORY5	Top	-249.9	-145.29	-6.66	-119.38	18
STORY5	Bottom	-249.9	-145.29	-6.66	-119.38	14.4
STORY4	Top	-287.66	-167.61	-8.19	-137.44	14.4
STORY4	Bottom	-287.66	-167.61	-8.19	-137.44	10.8
STORY3	Top	-308.9	-180.16	-9.34	-147.59	10.8
STORY3	Bottom	-308.9	-180.16	-9.34	-147.59	7.2
STORY2	Top	-318.33	-185.74	-10.1	-152.11	7.2
STORY2	Bottom	-318.33	-185.74	-10.1	-152.11	3.6
STORY1	Top	-320.69	-187.14	-10.49	-153.24	3.6
STORY1	Bottom	-320.69	-187.14	-10.49	-153.24	0

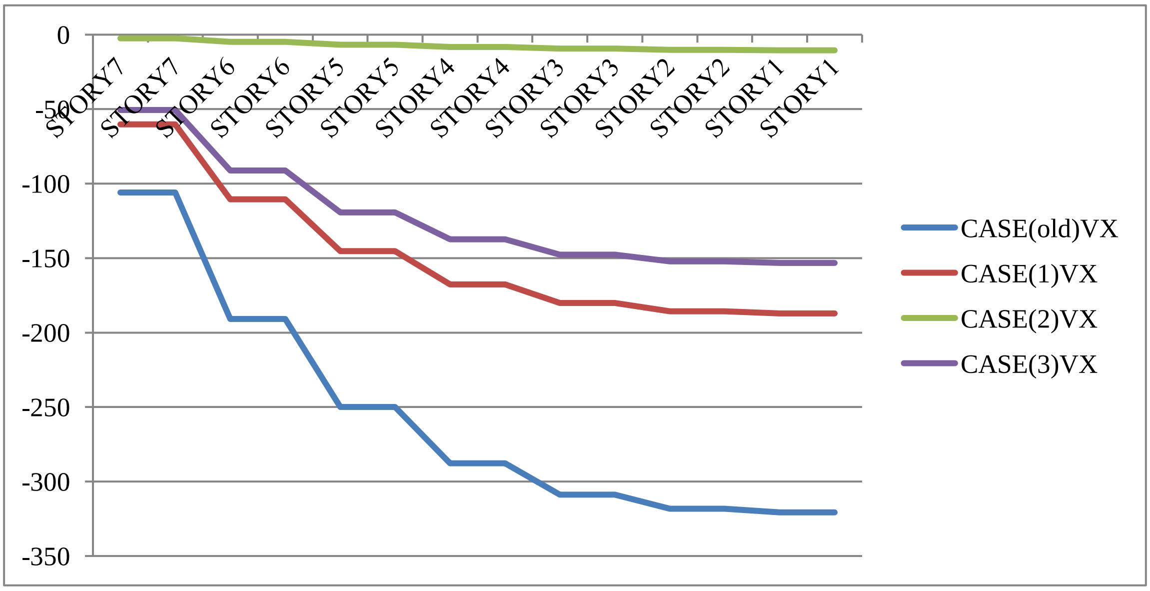


Figure No:-9.3 comparison for storey shear

9.4 .0 COMARISION FOR STEEL USED IN COLUMNS case1:- Columns with simple frame with column size 300x600mm

case2:- Columns frame with shear wall column size 300x600mm

case3:- Columns frame with base isolator column size 300x600mm

case4:- Columns frame Bracing rect steel box size 200x150x10

Table No 9.4(a) comparison for steel used in columns

STEEL REQUIREMENT(COLB/1)-Outer columns				
Storey	CASE(old)/S	CASE(1)/S	CASE(2)I	CASE(3)/B
STOREY7	19	27	15	15
STOREY6	21	27	15	14
STOREY5	25	30	15	14
STOREY4	35	35	20	15
STOREY3	53	40	30	16
STOREY2	85	47	30	21
STOREY1	52	32	37	14

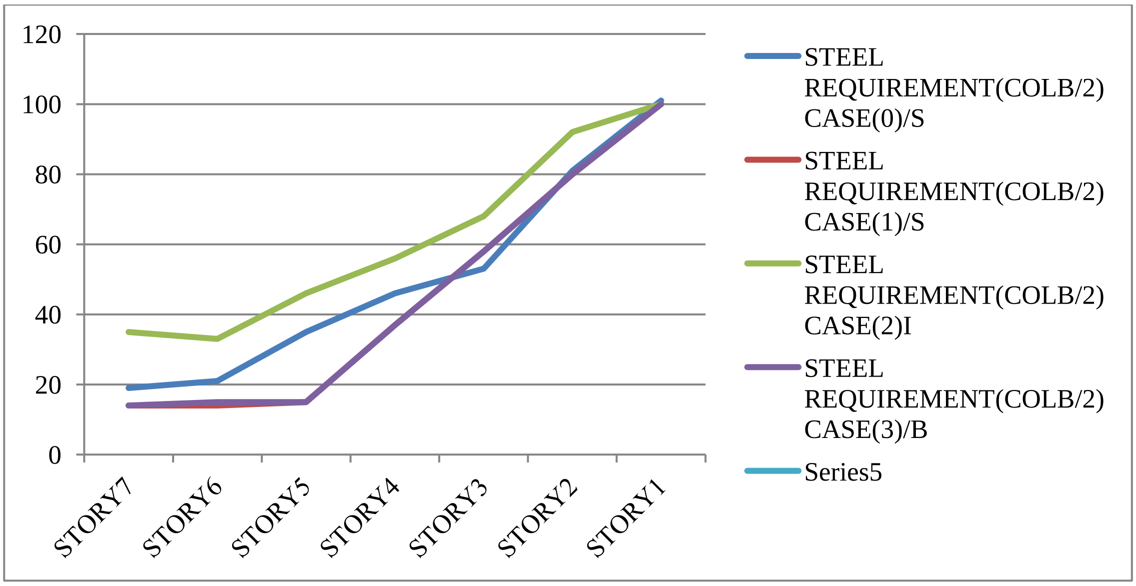


Figure no 9.4(a) Comparison of Steel Used In Columns)

Discussion: From comparison , we observe the following points

The steel requirement for a base isolated columns is lesser than others at corners.

(II) (Central columns column of centre raw column no B/2

case(old):- Columns with simple frame with column size 300x600mm

case1:- Columns frame with shear wall column size 300x600mm

case2:- Columns frame with base isolator column size 300x600mm

case3:- Columns frame Bracing rect steel box size 200x150x10

Table No9.4(b): comparison for steel used in columns

STEEL REQUIREMENT(COLB/2)inner columns				
Storey	CASE(old)/S	CASE(1)/S	CASE(2)I	CASE(3)/B
STORYE7	15	14	20	14
STOREY6	15	14	20	14
STOREY5	15	15	20	14
STOREY4	37	37	20	32
STOREY3	60	58	30	54
STOREY2	81	80	30	76
STOREY1	101	100	40	99

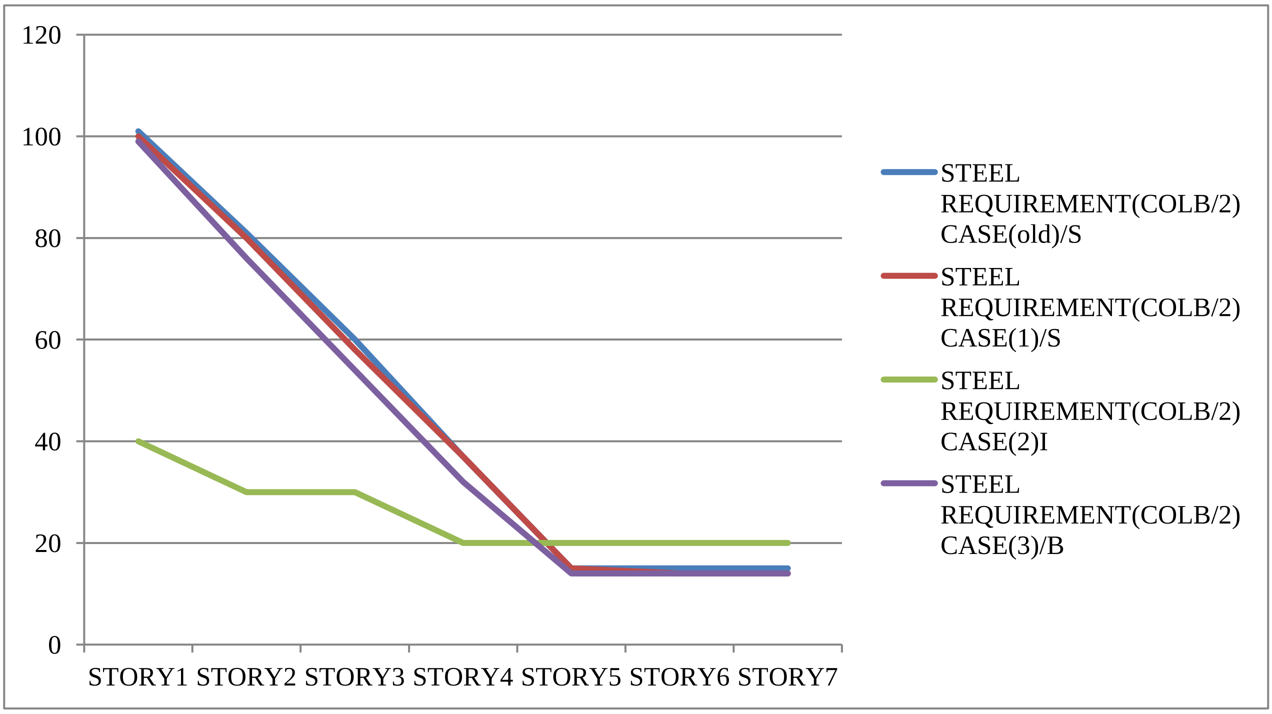


Figure no9.4(b) Comparison of Steel Used In Columns)

Discussion: From comparison , we observe the following points

(1) Internal columns have moreover similar steel for three cases.

(2) internal columns are less effected than the outer columns.

9.5Deflection Noted at a particular point (Point no 12 at Grid1)

Table No.9.5:- comparison of deflection at a critical point

Deflection(mm)at point 12 :-					Shear wall(0.3x4) 2xcase(2)
Story	CASE(old)/S	CASE(1)/S	CASE(2)I	CASE(3)/B	
STOREY7	27	50	8	23	40
STOREY6	24	48	7	18	35
STOREY5	21	39	6	14	28
STOREY4	17	30	5	10	20
STOREY3	12	20	4	6	12
STOREY2	7	11	3	3	6
STOREY1	3	3	2	1	2

Discussion: From comparison , we observe the following points

1 Shear wall is long cant liver more deflection(55mm) may reduced by increasing size.

2 well designed bracing provides economical deflection 23mm .

3 . Base isolation provides very less deflection only 8 mm as 23 mm in bracing

9.6 COMPARISON FOR PROJECT COST

The cost comparison is indicative and base on data collected from various agencies. The cost varies as per availability of material ,initial availability of funds, planning &requirement of systems overall serviceability, utility, for the structures having national importance.

(important bridge, important building having immediate occupancy level)

(1) Volume of RCC in shear wall =2x0.35x4x7x3.2x13000=8.1 say 10 lacs

Rate of Rs 13000/m3, including steel, concrete& placing

cost of shear wall

(2) (a)No of Isolators= 12xRs2.0 Laces/Each = 24 lacs

(b)Surrounding Isolation =60%of above = 5 lacs

(c) Flexible Services pipes installation 30% of (a) = 4 lacs

Total= a+b+c= 33.0 lacs

(3) Cost steel Bracings channels ISMC400@(48.46kg/M+2kg additional)

Weight of steel =7x4x2x6.8x50.46kg/mxRs70/kg=13.45lacs

Fabrication& Erection Rs 70000/MT

Table no 9.6 Comparison For Cost For Project Cost

COMPARISON				
PERIOD OF INVESTMENT	CASE(old) (RCC FRAME)	CASE(1) (SHEAR WALL)	CASE(2) (ISOLATION)	CASE(3) (BRACING)
INITIAL INVESTMENT	(RCC FRAME)	MODERATE 10.0lacs	HIGHER 33.0 laces	HIGHER THAN(1) 14.0laces
OVER INVESTMENT	(RCC FRAME)	MODERATE	HIGHER	MODERATE

Discussion: From comparison , we observe the following points

(1)The cost of Isolation of building is much higher than shear wall/ Bracing.

(2) The cost of Bracing is higher than the shear wall but lower than the Isolation.

Chapter 10

CONCLUSIONS AND FURTHUR SCOPE

10.1 CONCLUSIONS

The analysis of Retrofitting devices shear wall, Base Isolation and Bracing for seven story building were carried out to understand the seismic performance under seismic loading. Based on analysis following conclusion are drawn

1) From table No 9.1/Figure no 9.1 it is observed that . the Diaphragm deflection at 21.6 m height is 48mm in case of shear wall, 18.9mm in case X-bracing and only 7.64mm in case of base isolation Hence effectiveness of base isolation::X-bracing::shear wall is 6::2.5::1. Deflection for building is limited H/400(54mm) as per IS 1893 (Part I):2002.
2) From table No 9.2/Figure no 9.2 it is observed that storey deflection for shear wall & X-bracing is parabolic .It is maximum at mid height and minimum at top & bottom while in case of base Isolation story deflection is almost linear and negligible in comparison of Shear Wall & X-bracing(0.3,0.9and1.33mm respectively)
3) From table No 9.3/Figure no 9.3 Storey shear in VX - direction it is observed that Base shear value in case of base isolation is minimum/ negligible (10.49KN) and maximum187.14&153.24KN for shear wall& X-bracing respectively. Rate of Base shear reduction towards height is almost similar in all three case.
4) In short (From conclusion no 1, 2 & 3) Base Isolation Reduced lateral forces and displacement up to 75%(approx) & Safety is best with base isolation system
5) From table No 9.4(a) Figure no 9.4(b), it is observed that variation of steel requirement is more for outer columns (position wise as well as height wise)while it is negligible for inner columns. Hence it may be concluded that seismic forces changes drastically for outer columns while it remains almost same for inner columns..
6) FromTable9.6(Figure No9.6) and case study at base Isolation (page no 15) it is observed that initial costing is higher with base isolation system & need higher technical input .However, this system is economical for important structure like hospital, important Bridges , historical building, Atomic Reactor, Research& Technology centres.
7) Shear wall system is most economical and easy to construct with respect to Base Isolation & Bracing systems and gives good performance with monolithic structures

These conclusions may help in quantifying changes in different parameters and may be helpful in taking quick decisions during design of structures.

10.2 Scope of Future Work

As we have tried to study the different systems of seismic resistance device applicable for proposed structure or existing structure, there are some area where work can be done to attain better idea for design of seismic resisting structure

The Dampers are also a advanced seismic resisting devise These dampers act like the hydraulic shock absorbers in cars – much of the sudden jerks are absorbed in the hydraulic fluids and only little is transmitted above to the chassis of the car. When seismic energy is transmitted through them, dampers absorb most part of it, and thus damp the motion of the building.

Another approach for controlling seismic damage in buildings are use Tuned mass dampers (TMDs).These are passive control devices that are generally installed at the roof tops of buildings to control the responses of buildings produced due to wind or an earthquake. TMDs may be installed in other structures also, such as, flexible bridges (suspension/cable stayed bridges) to control the wind induced vibration

The design combination with these devise may be explored in a future study.

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