SEISMIC ANALYSIS OF CORE STRUCTURE

Fasil Mohi-ud-din [*1],Pavithra.C[2]

Fasil Mohi ud din[*1] M.Tech Structural Engineering Vellore Institute of Technology Vellore Tamil Naidu
Pavithra.C[2] Assistant Professor Department of Civil Engineering SRM University Tamil Naidu

Abstract

A comparative study of the structure with and without core was carried out which is based on linear analysis and non-linear static analysis of two prototype structural frame. The existing FEMA-356, ATC-1997a and the result from the previous research reports which were used in the analysis of the structure. This paper also aims at establishing comprehensive seismic design as well as characterizing earthquake response characteristics and failure mechanisms of structures through analytical investigations under large earthquake. To ensure the safety of the structure only shear wall technique has been widely used and not much has been done with respect of core structures. In this paper instead of shear wall, Concrete Core has been used; the displacement was calculated by default procedure and the yield of the structure in both the direction result in the development of plastic hinge under push over loads. The outcomes would lead to develop the most efficient and most economical frame which can withstand the major earthquake. It also lead to motivate advances in seismic design practice. The core provided within the structure resulted in the less lateral displacement and restricted the development of the plastic hinges at the initial stages hence making the design robust and acts as an energy dissipater.

Key words:- Core Structure, Peak Storey Shear, Time Period, Frequency, Displacement, Capacity Curve, Plastic hinge, performance point.

1. Introduction:

The Displacement of a tall building caused by horizontal forces due to wind or earthquake can be reduced by the provision of core. Figure 1, represents plan of a typical floor of a tall building, in which the core is composed of single solid concrete shaft. The core is the main lateral force-resistant component. The maximum horizontal displacement is limited by codes for the stability of the building and for the comfort of its occupants. Also, the codes limit the inter storey drift ratio, defined as the difference of drift in two consecutive floors divided by the vertical distance between them. The sum of the moments at the ends of a column at a floor level is a couple transferred, in the opposite direction, to the floor; the floor must be designed for the flexural and shear stress caused by the transfer. The moments transferred between the columns and the floors are mainly dependent on the inter storey drift ratio.

Pushover analysis is an incremental static analysis used to determine the force-displacement relationship, or the capacity curve, for a structure. The analysis involves applying horizontal loads, in a prescribed pattern, incrementally to the structure while keeping the gravity loads constant and pushing the structure latterly until the collapse in the structure will occur. Hence giving the plot between the base shear and the lateral displacement, leading to the development of plastic hinges. The intensity of the lateral load is slowly increased and the sequence of cracks, yielding, plastic hinge formation, and failure of various structural components are plotted in graphical and tabular format.

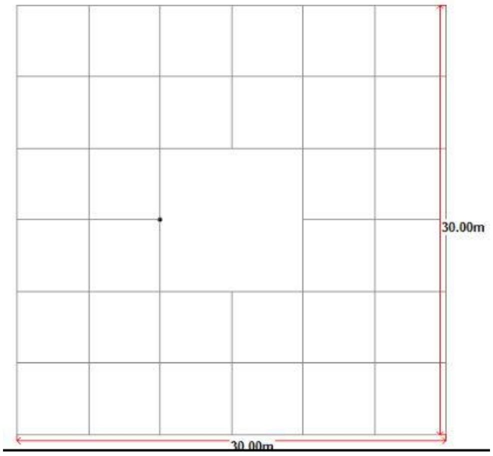

Figure 1 Without core

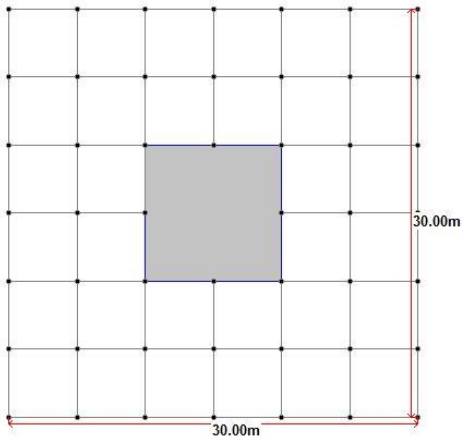

Figure 2 with core

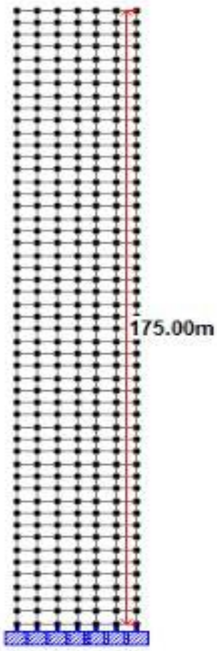

Figure 3 elevation

2. RESPONSE SPECTRUM (Linear Analysis):

The representation of the maximum response of idealized single degree freedom systems having certain period and damping, during earthquake ground motion. The maximum response is plotted against the undammed natural period and for various damping values, and can be expressed in terms of maximum absolute acceleration, maximum relative velocity, or maximum relative displacement.

3. Methodology:

The Response Spectrum Analysis was carried out for G+ 49 structures for the following dimensions.

Length of frame=30m

Breadth of frame=30m

Height of frame=175m

The core of 10mx10m and height of 175m is centrally placed and is running from the foundation level to the roof of the building.

Beam Dimension 350X550

Column Dimension 350X600

Grade of Concrete M60

3.1 Dead Load

Dead load of slab= 0.15X25 =3.75 kNm.

Floor finish = 1.5 kNm.

Wall Load = 0.23X19.23=4.41 kNm

3.2 Live Load

Live load = 3.5 kNm

Roof Live load = 1.5 kNm

3.3 Load Combinations:

1) 1.5(DL+lL)

2) 1.2(DL+ZL+EL)

3) 1.5(DL+EL)

4) 0.9DL* 1.5EL

3.4 Horizontal Seismic coefficient

$$A_h = \frac{ZISa}{2\,Rg}$$

Where, A_h = seismic design coefficient

Z = zone factor

R = Response Reduction Factor

S_a/g= Average response acceleration coefficient

$$A_h = \frac{0.16\ X\ 1.5\ X\ 0.27}{2\ X\ 5} = 0.00648$$

T_a = 0.075 $h^{0.75}$ for RC frame building

T_a = 3.60 sec

3.5 Base Shear

$V_b = A_h W$

Where V_b = Base Shear

W = Seismic Weight of Building

V_b=8179.04 kN (Base shear in X direction for the frame without core)

V_b=6116.62 kN (Base shear in Z direction for the frame without core)

V_b=17479.87 kN (Base shear in X direction for the frame with core)

V_b=17353.20 kN (Base shear in Z direction for the frame with core)

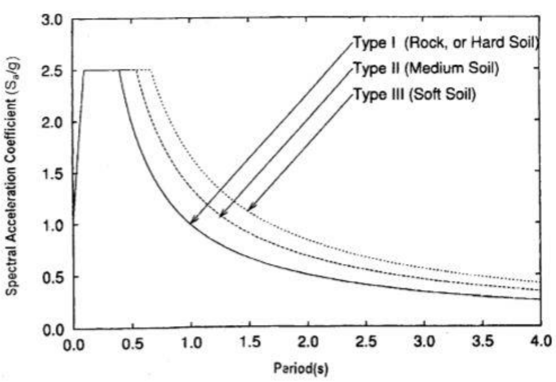

Figure 4

4. Response Spectrum Comparative Results:

4.1 Time period:

This Figure (4) depicts the time period variation of the two same structure's but of different structural configuration.

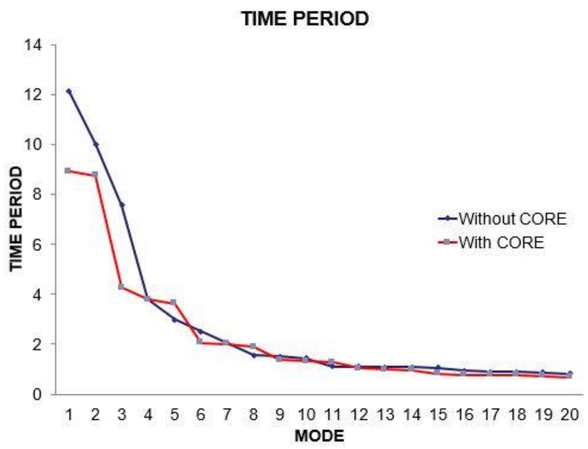

Figure 5

4.2 Frequency:

This Figure (6) depicts the frequency variation of the two same structure's but of different structural configuration.

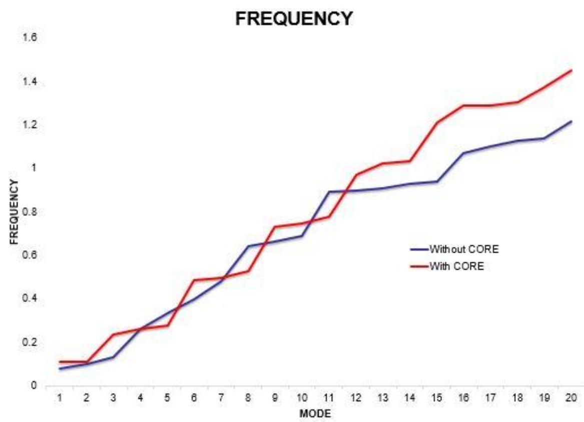

Figure 6

4.3 Spectral Acceleration:

This Figure (7) depicts the Spectral Acceleration variation of the two same structure's but of different structural configuration.

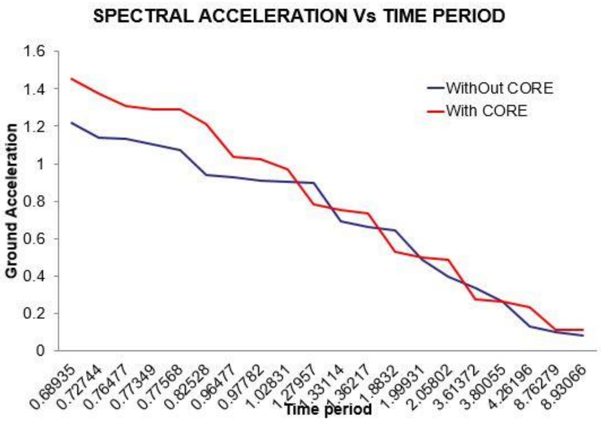

Figure 7

4.4 Peak Storey Shear:

This Figure (8) graph shows peak shear in X direction for without core structure.

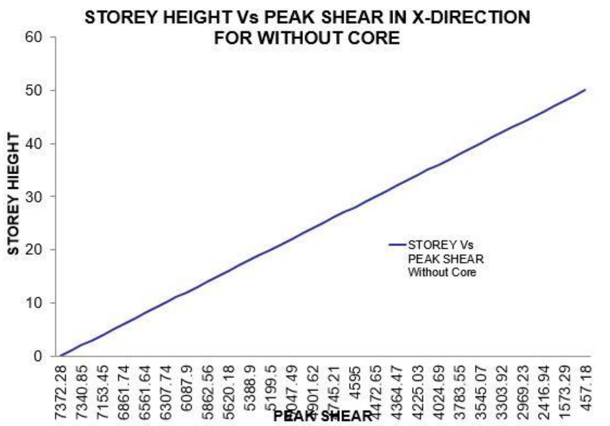

Figure 8

Figure (9) graph shows peak shear in X direction for with core structure.

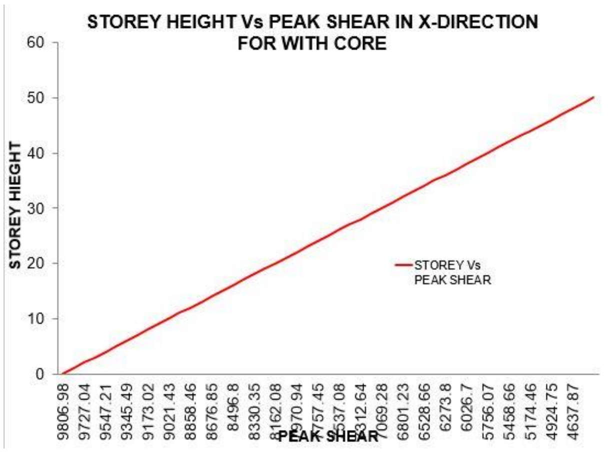

Figure 9

This Figure (10) graph shows peak shear in Z direction for without core structure.

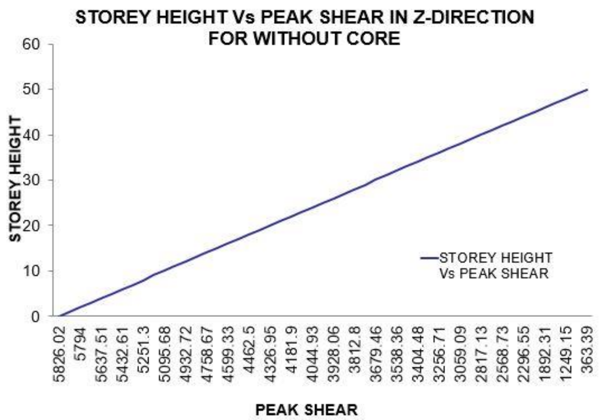

Figure 10

Figure (11) graph shows peak shear in Z direction for with core structure.

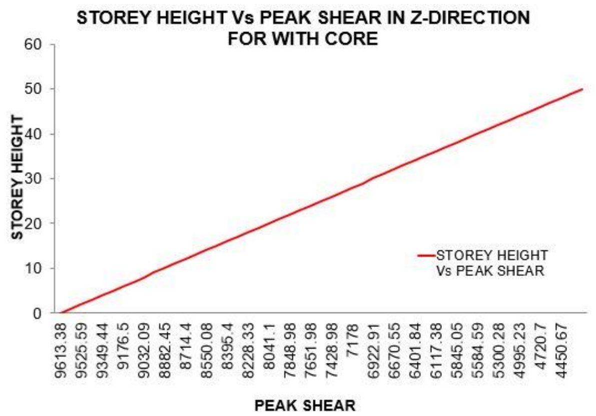

Figure 11

4.5 Mode Shape:

The participation factor as mentioned in the code IS1893:2002 should be minimum 90% or 50Hz since the mode shapes that were achieved showed the participation factor of more the 90% at earlier stages. Thus few mode shapes are depicted below for structure without core (Figure12) and with core (Figure13)

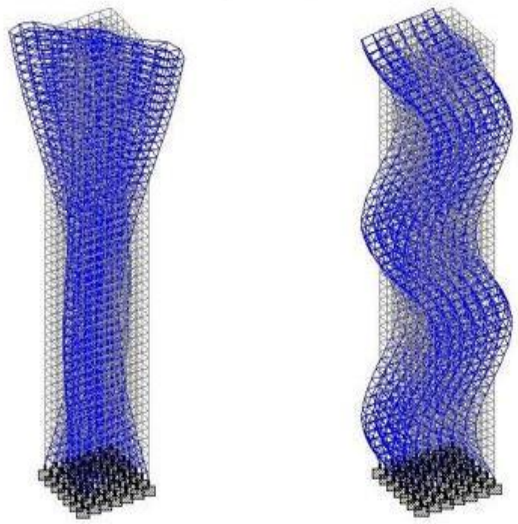

Figure 12

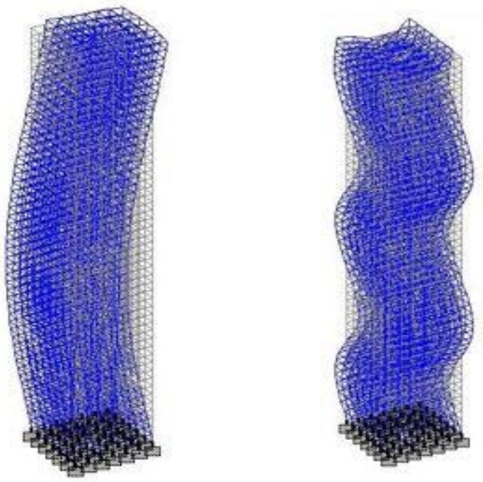

Figure 13

4.6 Displacement:

1. For Without Core

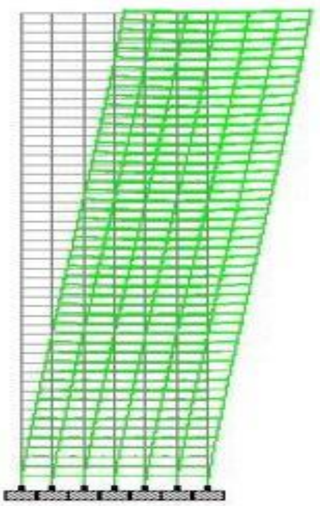

Figure 14

2. With Core

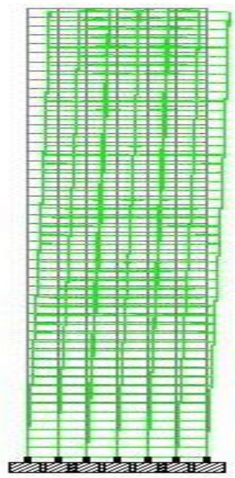

Figure 15

DIRECTION	X(mm)	Z(mm)	Resultant(mm)
Max X	1.22E 3	1.61E3	2.03E 3
Min X	-1.38E3	-1.67E3	2.21E 3
Max Z	1.13E 3	1.8E 3	2.16E 3
Min Z	-1.27E 3	-1.8E 3	2.25E 3

DIRECTION	X (mm)	Z (mm)	Resultant (mm)
Max X	**308.090**	312.119	439.395
Min X	**0.000**	0.000	0.000
Max Z	304.455	**312.120**	436.493
Min Z	0.000	**0.000**	0.000

5. STRESSES

5.1 Core Centre Stress

		Normal			Shear		
	Core	Sxx (N/mm²)	Syy (N/mm²)	Szz (N/mm²)	Sxy (N/mm²)	Syz (N/mm²)	Szx (N/mm²)
Max Fx	6565	**-0.425**	-5.133	-0.426	0.000	0.000	0.000
Max Fy	6565	-0.425	**-5.133**	-0.426	0.000	0.000	0.000
Max Fz	6565	-0.425	-5.133	**-0.426**	0.000	0.000	0.000
Max Sxy	6565	-0.425	-5.133	-0.426	**0.076**	0.077	0.000
Max Syz	6565	-0.425	-5.133	-0.426	0.076	**0.077**	0.000
Max Szx	6565	-0.425	-5.133	-0.426	0.000	0.000	**0.000**

5.2 Core Centre Principal Stress

	Principal			Von Mis	Direction S1			Direction S2		
	S1 (N/mm²)	S2 (N/mm²)	S3 (N/mm²)	Von Mis (N/mm²)	X	Y	Z	X	Y	Z
Max S1	**-0.425**	-0.426	-5.133	4.707	1.000	0.000	0.000	-0.000	0.000	1.000
Max S2	-0.425	**-0.426**	-5.133	4.707	1.000	0.000	0.000	-0.000	0.000	1.000
Max S3	-0.423	-0.425	**-5.135**	4.711	0.765	0.023	0.643	-0.643	0.002	0.766
Max Von Mis	-0.423	-0.425	-5.135	**4.711**	0.765	0.023	0.643	-0.643	0.002	0.766

5.3 Core Corner Stress

	Normal			Shear		
	Sxx (N/mm²)	Syy (N/mm²)	Szz (N/mm²)	Sxy (N/mm²)	Syz (N/mm²)	Szx (N/mm²)
Max Fx	**-2.645**	-9.966	-2.636	-0.077	-0.078	-0.294
Max Fy	-2.645	**-9.966**	-2.636	-0.077	-0.078	-0.294
Max Fz	-2.645	-9.966	**-2.636**	-0.077	-0.078	-0.294
Max Sxy	1.262	-0.888	1.259	**0.077**	0.078	0.294
Max Syz	1.262	-0.888	1.259	0.077	**0.078**	0.294
Max Szx	1.262	-0.888	1.259	0.077	0.078	**0.294**

5.4 Core Corner Principal Stress

	Principal			Von Mis	Direction S1			Direction S2		
	S1 (N/mm²)	S2 (N/mm²)	S3 (N/mm²)	Von Mis (N/mm²)	X	Y	Z	X	Y	Z
Max S1	**3.224**	0.814	0.424	2.627	0.021	1.000	0.021	0.712	-0.030	0.701
Max S2	-2.347	**-2.933**	-9.968	7.345	-0.702	-0.000	0.712	0.712	-0.016	0.702
Max S3	-2.347	-2.933	**-9.968**	7.345	-0.702	-0.000	0.712	0.712	-0.016	0.702
Max Von Mis	0.754	0.167	-9.379	**9.852**	-0.707	-0.000	0.707	0.707	-0.011	0.707

6. Push Over: (Non-Linear Static Analysis)

The existing building during the past earthquakes were severely damaged during 1976-2015.Structural failure was probably caused by a combination of factors that are not yet well understood. Earthquake source characteristics, site effects and structural vulnerability may be some of those factors. However, it is very difficult to assess the influence of each factor on structural failure. The push in the x-direction and in z-direction were generated by accelerating the frame in both the direction, which resulted in the formation of demand and capacity curve of the frame under Hard soil condition (Type-I). The simulated acceleration were then used to evaluate the structural nonlinear behavior of a reinforced concrete structure with and without core.

7. Methodology:

The Pushover Analysis was carried out for

G+ 5 structures for the following dimensions.

Length of frame =30m

Breadth of frame =30m

Height of frame =18m

Size of core

Length of frame =10m

Breadth of frame =10m

Height of frame =18m

The core is centrally placed and is running from the foundation level to the roof of the building.

Beam Dimension 350X450

Column Dimension 350X550

Grade of Concrete M20

7.1 Dead Load

Dead load of slab = 0.15X25 =3.75 kNm.

Floor finish = 1.5 kNm.

Wall Load = 0.23X19.23=4.41 kNm

7.2 Live Load

Live load = 3.5 kNm

Roof Live load = 1.5 kNm

7.3 Load Combinations:

1) 1.5(DL+lL)
2) 1.2(DL+ZL+EL)
3) 1.5(DL+EL)
4) 0.9DL* 1.5EL

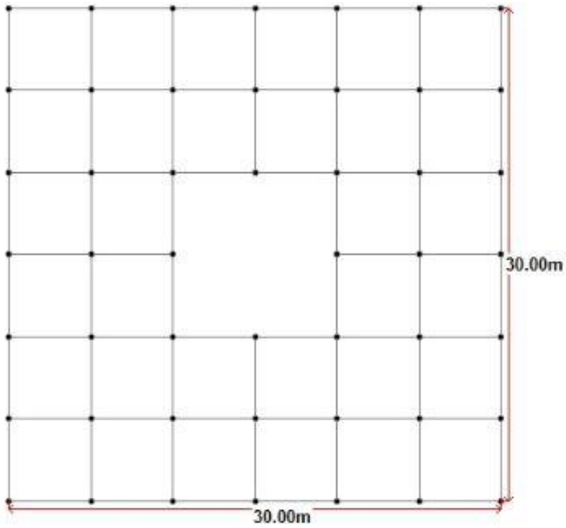

Figure 16 Without core

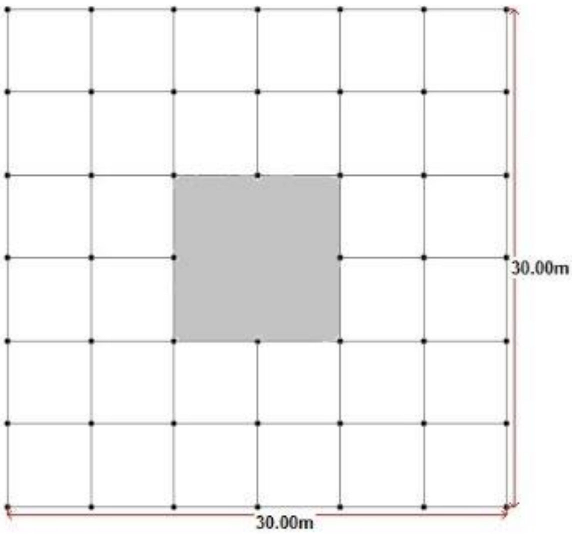

Figure17 Withcore

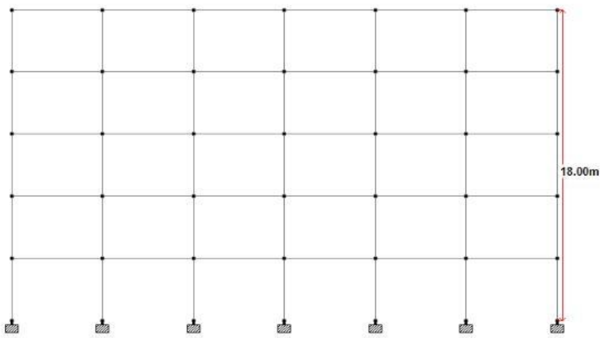

Figure 18 Elevation

8. Push Over Comparative Results:

8.1 Pushover Analysis result in X-direction

Plain Structure

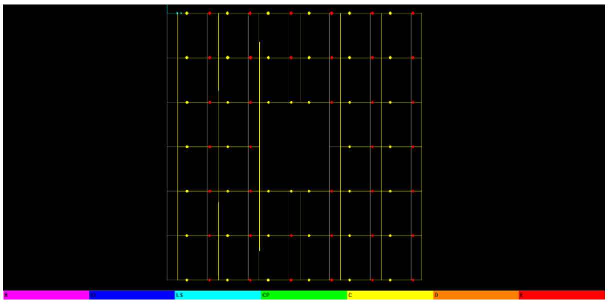

Figure 19

Core Structure

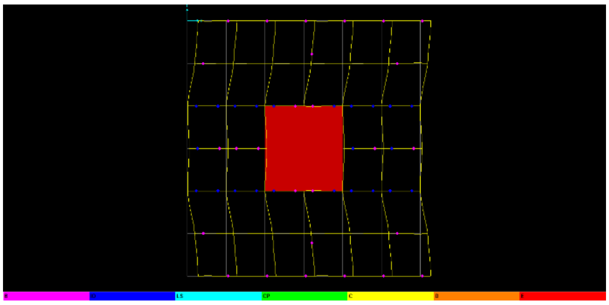

Figure 20

It is evident from the above Figure's that the displacement in the lateral X-direction has been reduced by placing the concrete core into the center of the structure and development of the Plastic hinges has been restricted in the initial stages only. As it has been already proved by few research papers earlier that in core structures during earthquakes the most part which gets damaged is the base of the core and the components around it periphery.

8.1.1 Elevation:

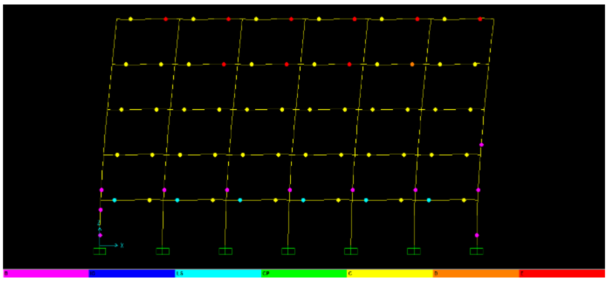

Figure 21

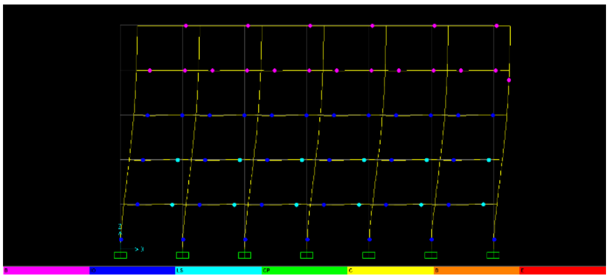

Figure 22

While Differentiating the above two figure's it is evident that the building without the structure lateral resisting system has shown the hinge formation beyond collapse stage and the structure with lateral resisting system has relatively performed better. Since the hinge formation is restricted in initial stages only.

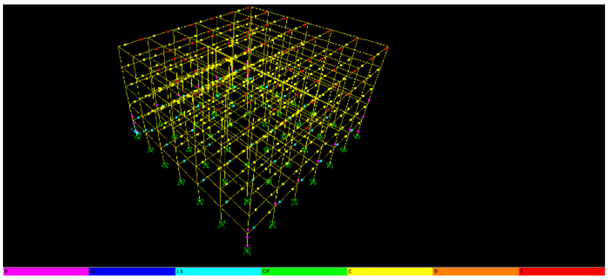

Figure 23

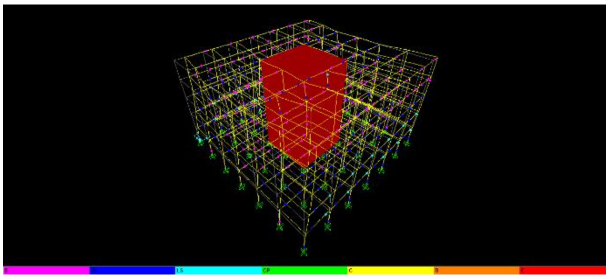

Figure 24

The above image depicts the 3D View of the Structure with and without lateral resisting system.

8.1.2 Pushover Curve:

8.1.2.1 Resultant Base Shear vs. Monitored Displacement:

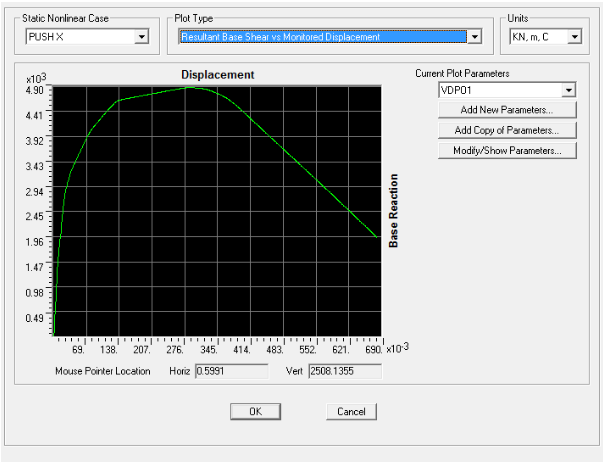

Figure 25

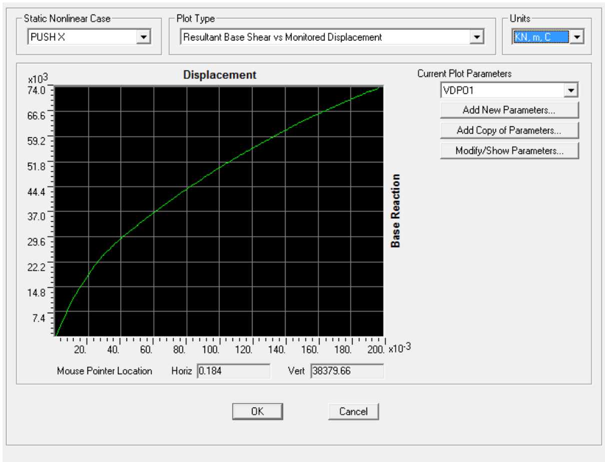

Figure 26

As explained by the (ATC,1997a) guidelines if the performance of the structure is higher it results in less loss of life. As depicted from the pushover analysis from the above structures. It revelead that the structure in whch the core has been placed performed relatively better with respect to the structure without lateral resisting system. The pushover curve shows that the structure can withstand the base shear of 4900kN and when moved to the lateral displacement of 0.28m. While as the structrure with lateral resisting system can withstand the base shear of 74000kN when moved to a lateral displacment of 0.2m.

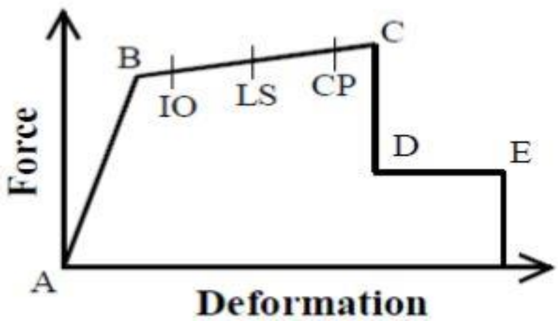

Figure 27

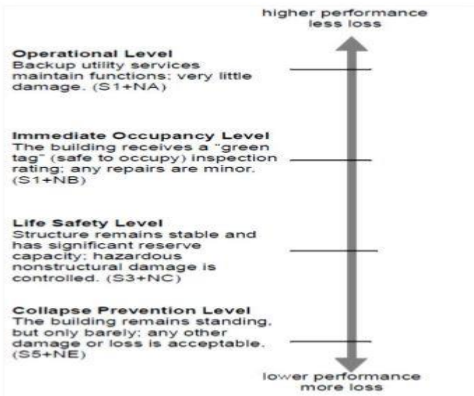

Fig. 3.2 Building Performance Levels (ATC, 1997a)

Figure 28

8.1.2.2 ATC Capacity Spectrum:

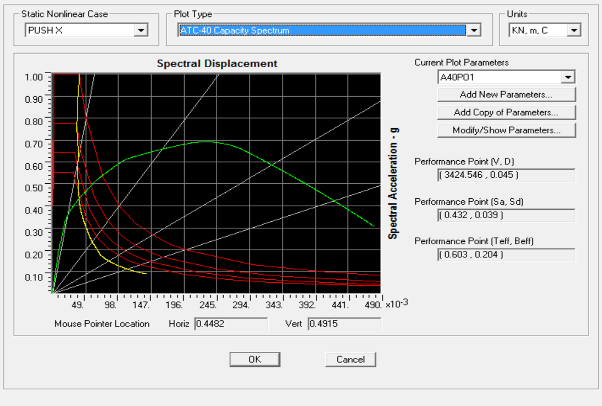

Figure 29

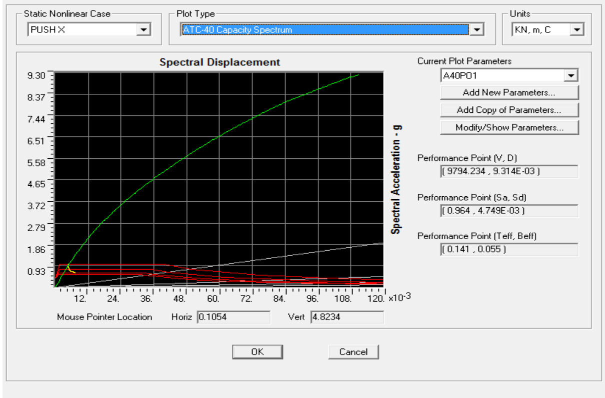

Figure 30

Performance point as shown in the structure with lateral resisting system is 9794.2kN with lateral displacement of 0.009m while as the structure without this system is help the base shear capacity of 3424.5kN up to the displacement of 0.045m.

8.1.3 Hinge Development:

The below fig. depicts the beam and column hinge at the collapse stage and the path traversed by the hinge to its ultimate stage.

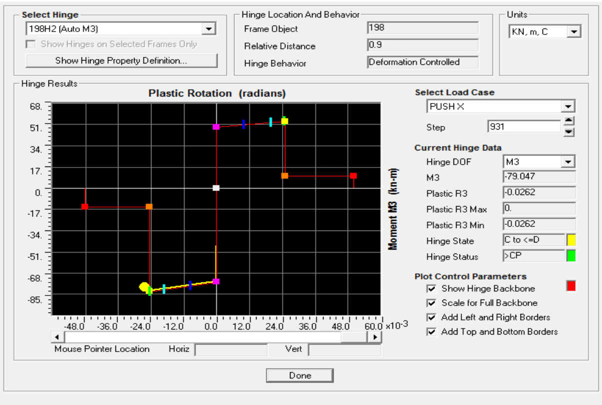

Figure 31

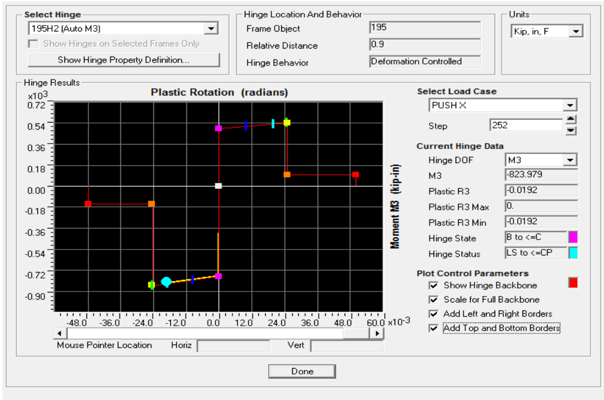

Figure 32

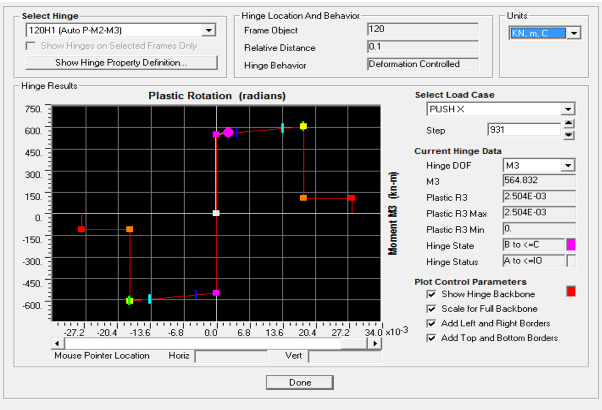

Figure 33

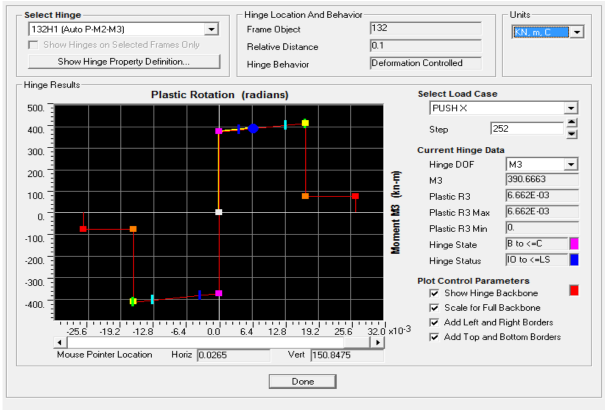

Figure 34

8.2 Pushover Analysis result in Y-direction

Plain Structure

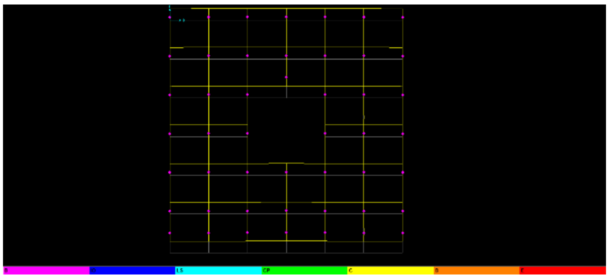

Figure 35

Core Structure

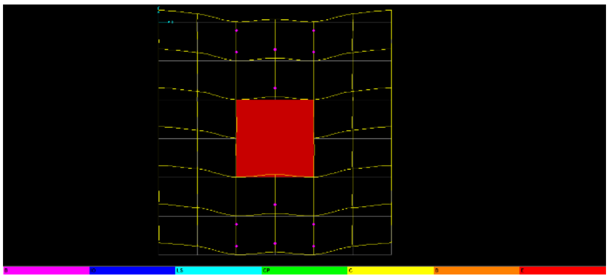

Figure 36

The above fig. shows the restrained displacement in the Y-Direction due to induction of core in the structure.

8.2.1 Elevation:

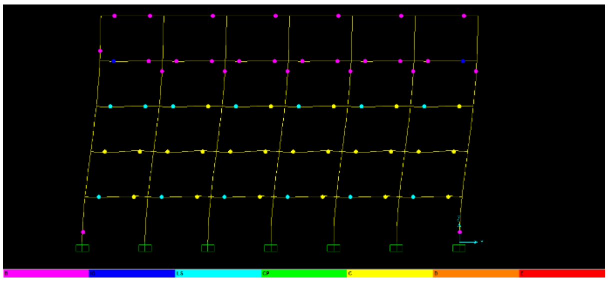

Figure 37

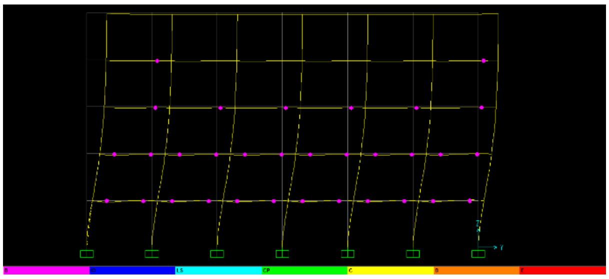

Figure 38

8.2.2 Pushover Curve:

8.2.2.1 Resultant Base Shear vs. Monitored Displacement:

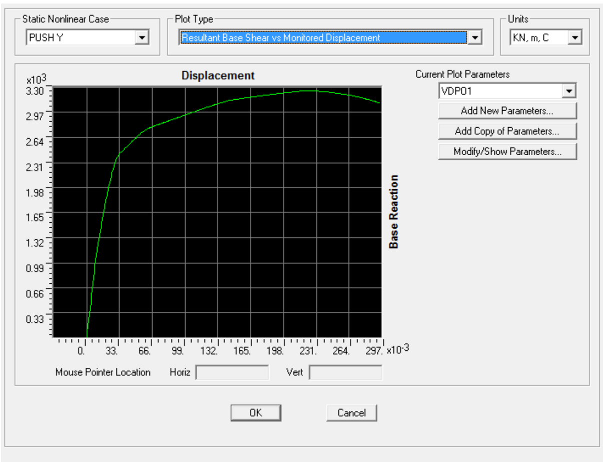

Figure 39

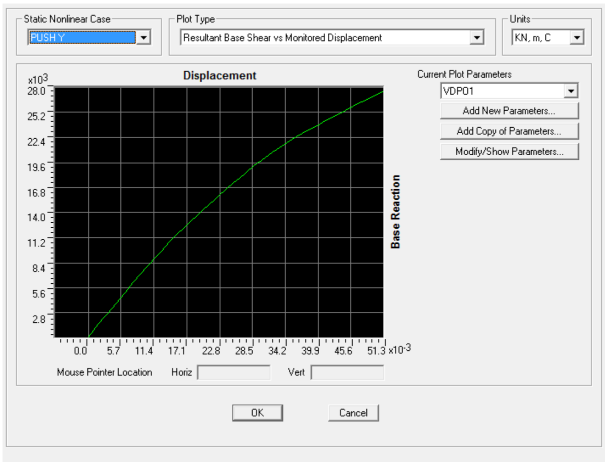

Figure 40

The pushover curve shows that the structure can withstand the base shear of 3300kN and when moved to the lateral displacement of 0.23m. While as the structrure with lateral resisting system can withstand the base shear of 28000kN when moved to a lateral displacment of 0.05m.

8.2.2.2 ATC Capacity Spectrum:

Performance point as shown in the structure with lateral resisting system is 9226.2kN with lateral displacement of 0.012m while as the structure without this system is help the base shear capacity of 2691.0kN up to the displacement of 0.054.

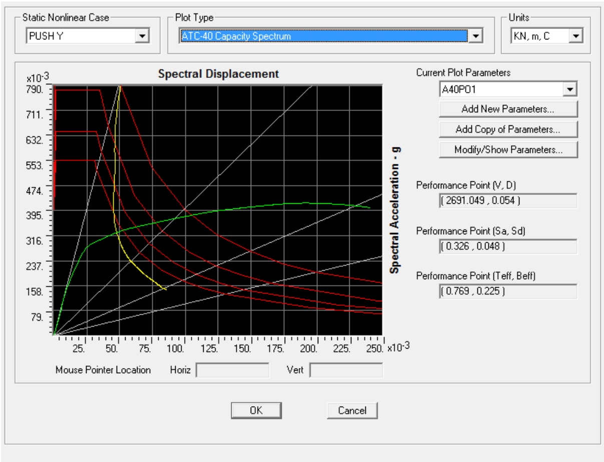

Figure 41

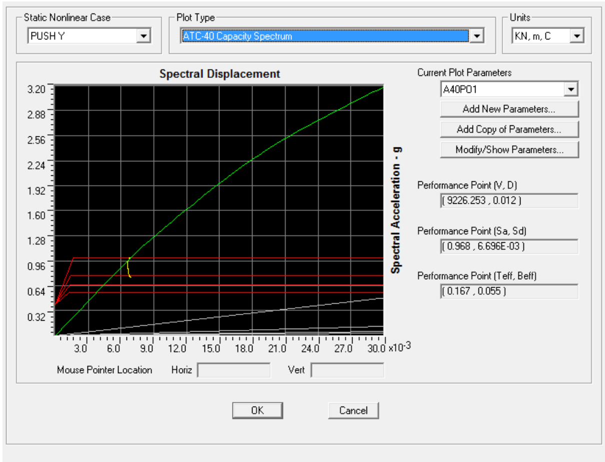

Figure 42

9. CONCLUSION:

9.1 Response Spectrum:

1) Time Period

The Percentage reduction and time period for the two structures with two set of configuration with and without core is 26.31%.

2) Frequency:

The percentage increase as computed for two structures is 19.22%

3 Base shear:

The Percentage in base in the base shear computed shows the increase of 113%

4) Displacement

The percentage displacement reduction in core structure when compared to without core it has reduced up to 78.35%

Hence the result deduced from the Response Spectrum analysis shows the structure with core more stable against earthquake.

9.2 Pushover:

1) Capacity Curve

In X-direction, the Percentage of Base Shear got increased up to 1410% by the induction of lateral Resisting system.

In Y-direction, the Percentage of Base Shear got increased up to 748.4% by the induction of lateral Resisting system.

2) Performance Point

In X-Direction, the Percentage of Shear Capacity increased by 186% as compared to the non-core structure.

In Y-Direction, the Percentage of Shear Capacity increased by 242.8% as compared to the non-core structure.

3) Plastic Hinge Development

In the case of Structure Without core the collapse hinge Development started at the earlier stage while as the structure with core resisted the formation of collapse hinge up to longer stage before the ultimate failure of the structure.

REFERENCE:

1. ATC-2006, Next-Generation Performance-Based Seismic Design Guidelines: Program Plan for New and Existing Buildings, FEMA 445, Federal Emergency Management Agency, Washington, D.C.
2. ATC-1997a, NEHRP Guidelines for the Seismic Rehabilitation of Buildings, FEMA 273 Report, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, D.C.
3. Bureau of Indian Standards (IS 1893-2002).
4. Chung- Yue Wang and Shaing-Yung Ho (2007). “Pushover Analysis for Structure Containing RC Walls”. The 2nd International Conference on Urban Disaster Reduction.
5. Computer and Structures, Inc. (CSI) (2000). SAP2000/NL-PUSH software, version 7.40. Berkeley.
6. Hassan .M & Driver G.O. “Column demands in steel plate shear walls with regular perforations using performance-based design methods.”
7. Jian-GN & Fan-Min Bu “Effective stiffness of composite shear wall with double plates and filled concrete”.
8. Kabeyasawa, T.(1993) "Ultimate-State Design of Wall-Frame Structures," Earthquake Resistance of Reinforced Concrete Structures.
9. Mehmet.I, Baytan.H . (2006) “Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings”. Department of Civil Engineering, Pamukkale University, 20070 Denizli, Turkey.
10. Marc.B, Nebojsa, BOVIC.O, Bernard, “Tests on reinforced concrete low-rise shear walls under static cyclic loading.”
11. Murthy C.V.R & Sudhir.K.Jain (2000), “Beneficial Influence of Masonry Infill walls on seismic performance of the RC frame Buildings”. (Asst Prof at IIT Kanpur)
12. Patil S.S, Ghade S.A , Konapure C.G, Gadhe C.A. “Seismic Analysis of the High Rise building by Response Spectrum method”
13. SiamakSattar , “University of Colorado Boulder .Influence of Masonry Infill walls and other Building Characteristics on Seismic Collapse of the Concrete Frame Building”
14. Wen-I Liao, JianxiaZhong , Lin C.C. , Y.L. Mo and Chin-HsiungLoh “Experimental studies of high seismic performance shear walls.”