Civil and Structural Engineering

Experimental and Theoretical

Standpoint of Creep Adsorption

Highlights {

Pushover Analysis of an OMRF

Seismic Vulnerability Assessment

Discovering Thoughts, Inventing Future

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By Oyati, E. N

Auchi Polytechnic, Auchi

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GJRE-E Classification: FOR Code: 090599

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I. Introduction

The world faces today the big challenge of traffic accidents that harvest annually millions of human lives (Muhammad, 2013). The consequences of these traffic accidents do not only affect the victims or their families, but extend to the impact the community and its progress (Muhammad, 2013). Pedestrian bridges are structures made for allowing pedestrians to cross a street/road/highway without being exposed to the risks of car accidents. A pedestrian bridge is any structure that removes pedestrians from vehicle roadway (Muhammad, 2013).

The first pedestrian bridge in Nigeria was a steel structure erected at Idumota cenotaph on Lagos Island (The Guardian, 2015). However, according to the Guardian newspaper, two such concrete bridges were also constructed: one in Iddo railway terminals across the road and the second was from Oyingbo to Otto near the old Leventis mainland hotel. The two bridges were planned towards the 1960 independence celebration. The construction work was carried out by Taylor Woodrow Construction Company (The Guardian, 2015). It was a major event on its own in those days especially considering the swampy terrain that the bridges were required to cross through. With the advent of the third National Development Plan (1975-1980), reinforced concrete bridges on piles and prefab deck were constructed over Apapa-Oshodi expressway and the Agege Motor Way at Ikeja. A bridge is a structure that provides passage over an obstacle such as valley, rough terrain or body of water by spanning those with natural or manmade materials (Newman, 2003; Mosley and Bungey,1999; Jeswald, 2005).

According to Mugu (2004) a footbridge or pedestrian bridge is principally designed for pedestrians and in some cases cyclists, animal traffic and horse riders rather than vehicular traffic. Recently the Lagos State Government erected a multi-functional pedestrian structure at Oshodi (The Guardian, 2015). The current governor of Lagos State, Akinwumi Ambode, has approved the construction of pedestrian bridge at Berger area of the State to give room for easy crossing by pedestrian of the ever busy Lagos- Ibadan expressway (P M News, 2015). In Benin City, Edo State of Nigeria, there was a pedestrian steel bridge constructed at close proximity to the University of Benin main gate but was dismantled because of the dualisation of the road by the Edo State Government. Types of pedestrian bridge include: simple suspension, clapper, moon, step stone and zig zag bridge

Increasing rate of accident at the hostels' gate of Auchi Polytechnic is worrisome. This involves either two or more vehicles or at times two or more motor bikes. The fatal ones always attract the attention of the Federal Road Safety Corp (FRSC) who needs to evacuate the vehicles and the injured in order to allow the free flow of traffic. The main victims of hit and run by vehicles and bike riders especially at night have been the students living on and off campus of Auchi Polytechnic, Auchi. Thus this development necessitates the design and construction of a pedestrian bridge across the Auchi-Benin highway.

II. Materials and Methods

a) Study Area

This study focuses on the design of pedestrian bridge across the Auchi-Abuja Highway in front of Auchi Polytechnic, Auchi main entrance gate.

b) Design consideration, calculations and analysis

i. Soil Test

The following geotechnical parameters were determined as follows:

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Percentage soil sample retained (%) = $\dfrac{mass\,retained}{total\,mass}$ x $\dfrac{100}{1}$ [1]

Percentage passing (%) = $\dfrac{total\,percentage\,retained}{\%\,retained\,of\,each\,sieves}$ [2]

Coefficient of uniformity (cu) = $D30/_{D60}$ x D10   [3]

Coefficient of uniformity (cu) = $D60/_{D10}$ [4]

ii. *Specific Gravity Test (Gs)*

Table 1: Specific Gravity Result at 0.5m & 1m: Data Sheet for Specific Gravity by Density Bottle

S/N	Observation and Calculations	Determination	No
		1	2
1	Density	4267.0	4331.0
2	Weighty of Empty Bottle (M_1) (g)	26.6	26.6
3	Weighty of Bottle +Sample (M_2) (g)	56.6	56.6
4	Weighty of Bottle + Sample H_2O (M_3) (g)	94.0	94.1
5	Weighty of Bottle + Sample + H_2O (M_4) (g)	77.6	76.6
6	M_2-M_1 (g)	30.0	30.0
7	M_4-M_3 (g)	13.6	13.5
8	Gs	2.2	2.2
Result	Density Gs = 5	2.2	2.2

Field data, 2016

Given:

Gs = (M_2-M_1)/ (M_2-M_1)-(M_4-M_3)

iii. *Density Test*

Table 2: Calculation and Results: Data Sheet for Dry Density by Core Cutter Method

S/N	Observation and Calculations	Determination	No
		1	2
1	Core Cutter No	501.0	502.0
2	Internal Diameter (cm)	10.0	10.0
3	Internal Height (cm)	13.0	13.0
4	Mass Of Empty Core Cutter (M_1)g	900.0	900.0
5	Mass Of Core Cutter With Soil (M_2)g	2900.0	2600.0
6	Mass Of Wet Soil M= M_2-M_1	2000.0	1700.0
7	Volume Of Cutter (V) (cm³)	1021.0	1021.0
8	Moisture Content	0.1	0.2
9	Bulk Density = Wt of Soil / Vol of Cutter (g/cm³)	1.96	1.67
10	Dry Density (g/cm³)	1.75	1.45

Field data, 2016

Volume of cutter (V), mass of wet soil (M), Bulk density and dry density were computed as follows:

$$V = \pi r^2 H$$
    [5]

$$M = M_2\text{-}M_1$$
    [6]

Bulk density (Bd):

$$Bd = \frac{M_2 - M_1}{\pi r^2 H}$$ [7]

Dry density $= \dfrac{\text{Bulk density}}{1 + \%moisture}$ [8]

iv. *Shear Strength of Soil*
   First Test Run 10kg = 66 Div.
   Second Test Run 20kg = 95 Div.
   Third Test Run 30kg = 115 Div.
   Shear Stress at Failure
   Test Run 1 (10kg)
   Shear at Failure = 0.577 x 66 = 38KN/m²
   Test Run 2 (20kg)
   Shear Stress at Failure = 0.577 x 97= 56KN/m²
   Test Run 3 (30kg)

Shear Stress at Failure = 0.577 x 115 = 66 KN/m²

Normal Stress

Test Run 1 (10kg)

Normal Stress = 21.8 + 10 x 2.75 = 49 KN/m²

Test Run 2 (20kg)

Normal Stress = 21.8 + 20 x 2.75 = 77 KN/m²

Test Run 3 (30kg)

Normal Stress = 21.8 + 30 x 2.75 = 104.3 KN/m²

Table 3: Shear Strength Failure Result

Test No.	Load	Shear Stress at Failure (KN/M2)	Normal Stress (KN/M2)
1	10	38.0	49.0
2	20	56.0	77.0
3	30	66.0	104.3

Field data, 2016

Table 4: Shear Strength at Depth 1m

Time	Mass 1	Maxx 2	Mass 3
(Seconds)	10 kg	20 kg	30 kg
5			
10			1
15			1
20			1
30			1
60			1
90	1		13
120	2	29	56
150	4	46	75
180	16	54	87
210	30	61	93
240	40	71	93
270	47	76	106
300	50	82	105
330	52	86	109
360	53	87	112
390	57	92	115
420	61	97	
450	62		
480	65		
510	65		
540	66		
570			
600			
630			
660			
690			
720			

Field data, 2016

v. *Bearing Capacity Computation*

Length = 1800mm; Breadth = 1200mm; Depth = 1000mm

$$Qu = 1.3CNC + rDfNq + 0.4rBNr \qquad [9]$$

Qu = 1.3 x 8 x 37.2 + 9.8 x 1 x 22.5 + 0.4 x 9.8 x 1.2 x 19.7

Qu = 386.88 + 220.5 + 92.67

Qu = 700KN/m²

Where factor of safety = 3

Allowable bearing capacity = 700/3 = 233KN/m²

Net allowable load = 233 x 1.8 x 1.2= 503KN.

vi. *Consolidation Test*

For 2kg Load

$$\text{Stress } \tau = \frac{force\,x\,beam\,ratio}{area} \qquad [10]$$

Beam ratio = 10.00

Diameter of sample = 5cm =0.05m

$$\text{Area} = \frac{\pi d^2}{4} = \frac{3142 x 0.05^2}{4} = 1.96 x 10^{-3} m^2$$

$$\text{Stress } \tau_1 = \frac{2 x 9.81 x 10 x 10^{-3}}{1.96 x 10^{-3}}$$
$$= 100.102 KN/M^2$$

For 4kg Load

$$\text{Stress } \tau_2 = \frac{4 x 9.81 x 10 x 10^{-3}}{1.96 x 10^{-3}}$$
$$= 200.204 KN/M^2$$

For 6kg Load

$$\text{Stress } \tau_3 = \frac{6 x 9.81 x 10 x 10^{-3}}{1.96 x 10^{-3}}$$
$$= 300.306 KN/M^2$$

For 8kg Load

$$\text{Stress } \tau_4 = \frac{8 x 9.81 x 10 x 10^{-3}}{1.96 x 10^{-3}} = 400.408 KN/M^2$$

For 10kg Load

$$\text{Stress } \tau_4 = \frac{10 x 9.81 x 10 x 10^{-3}}{1.96 x 10^{-3}}$$
$$= 500.510 KN/M^2$$

Calculation for Coefficient of Consolidation under Stress

Ao =1.594; Af = 1.678; T50 = 0.018; A50 = 1.640

Note: These values were read off from the consolidation graph.

$$A_s = \frac{A_f - A_o}{2} \qquad [11]$$

Determination of Cv

$$Cv = \frac{0.20 H^2}{t50} \qquad [12]$$

where H = ½ (H1 + H2)

$$Cv = 0.20 \frac{(0.0175)^2}{(0.018)^2} = 0.19 m^2/mins$$

Determination of Co-efficient of Compressibility (Av)

$$Av = \frac{\Delta e}{\Delta p} \qquad [13]$$

$$= \frac{0.0565}{200.204 - 100.102}$$
$$= 56 x 10 e^{-4} m^2/KN$$

Determination of Mv

$$Mv = \frac{av}{1+ef} \qquad [14]$$

$$= \frac{5.6\, x 10 e^{-4}}{1+ef} = 3.7 \times 10 e^{-4} m^2/KN$$

Initial Saturated Density (Psat)

$$\text{Psat} = \frac{Gs+e}{1+e} x\, pw \qquad [15]$$

$$= \frac{2.59+0.129}{1+0.129} x 1000$$
$$= 2408\ kg\ /\ m^3$$

Determination of K

$$K = \frac{Mv.cv.9.81}{1408\ x\ 62} \qquad [16]$$

Table 5: Consolidation Result

ΔH	Δe = 0.0565 ΔH	e = e - Δe	h = Ho - ΔH	Effective Stress
0	0.00000	0.513	20.00	0.0000
0.86	0.04859	0.464	19.14	100.102
1.16	0.06554	0.448	18.84	200.204
1.43	0.08079	0.432	18.57	300.306
1.59	0.08984	0.423	18.41	400.408
1.69	0.09549	0.448	18.31	500.501

Field data, 2016

c) *Design of Structural Elements*

   i. *Live Load for Footbridge*

   For loaded length in excess of 30m

   Live load, $qk = k \times 5.0 KN/m^2$ [17]

   Where,

   K= nominal HA UDL for appropriate
   loaded length (in KN/m) [18]
   $\frac{\text{}}{30 \text{ KN/m}}$

HA value for loaded length (32.4m) = 29.1 KN/m

   Therefore,
   K=$\frac{29.1 \text{ KN/m}}{30 \text{ KN/m}}$ = 0.97

   But $qk = k \times 5.0$ KNlm²

Therefore, $qk = 0.97 \times 5.0$ KNlm²= 4.85 KN/m²

   ii. *Steel Plate for Treads*
Assume 300mm x 2400mm size

Plate Loading
Dead load from plate
=25.55kg/m²x0.3mx2.4m+25.55kg/m²x0.147mx2.4m
=27.41kgx10=274.1Nx10⁻³=0.274KN

Characteristic impose load =2.4m x0.3m
x4.85kN/m²=3.49KN

Impose load on riser/tread=3.49x22=76.78KN

Design load, n=1.4gk+1.6qk

=1.4x0.274+1.6x76.78=123.23KN

Bending moment

   $BM_{max} = WL/8$ [19]

=123.23x0.3/8 =4.62KNM
For short span girder
Span/depth=12
Span/depth=15
2390mm/depth=15
Depth=2390mm/15=159mm≈160mm
Since d=160mm>150mm; take p_y=225N/mm²

   Area of flanges, Af=M/d x p_y [20]

=4.62x106/160 x 225
4620000/36000=128.33mm²
Width of the flanges is within the range of 0.5d
=0.5x160mm=80mm
128.33/80=1.604; assume 2mm
Assume a plate size 80mmx2mm
Area=80x2=160mm²>128.33mm²
The section chosen for the plate girder flanges as OK, the plate can be used.
Adopt the section 80mmx2mm

For the Web

   $T \geq d/20$ [21]

$T \geq$ 160/20=1.333
Since T is a little bit small use the dimension of the flanges for the web 80mmx2mm

Section Classification
Flanges
T=160mm; p_y=225N/mm²
But

   $ɛ=(275/py)^{1/2}$ [22]

$ɛ=(275/225)^{1/2}$=1.11
b=$\frac{80-2}{2}$ =39

T=160mm
b/T=39/160=0.243
For welded section
b/T=13ɛ
b/T=13x1.11 = 14.43>0.24
Therefore, the flanges are semi compact

Serviceability Deflection under Imposed Load

   $\delta = \frac{050}{384} \times \frac{wl^4}{EI}$ [23]

W = 1.0gk + 1.0qk
   = 13.814+38.39=52.204KN
Where, E=205x10⁹N/mm²

$I = 4010 cm^4 = 4010 \times 10^4 mm^4$

$\delta = \dfrac{5}{384} \times \dfrac{52.204 \times 10^3 N \times (7670)4mm4}{7670mm \times 205 \times 10^9 N/mm2 \times 4010 \times 104mm4}$

$= \dfrac{9.0334 \times 10^{20}}{2.4212 \times 10^{25}}mm$

$= 3.7309 \times 10^{-5} mm$

But $\delta <$ span/360

$= 3.7309 \times 10^{-5} mm < 7670mm/360$

$= 3.7309 \times 10^{-5} mm < 21.30mm: Mc > BM_{max}: Ok$

Moment Capacity

$$Mc = pySx \qquad [24]$$

$Sx = 1010 cm^3 = 1010 \times 10^3 mm^3$

$Mc = 275 N/mm2 \times 1010 \times 10^3 mm^3$

$= \dfrac{277750000 Nmm}{10^6}$

$= 277.75 KNm > 253.76 Knm$

Serviceability deflection under imposed load

$W = 1.0gk + 1.0qk$

$= 25.23 + 75.60$

$= 100.83KN$

$\delta = \dfrac{5wl4}{384EI}$ (from eq. 23)

Where $E = 205 \times 10^9 N/mm^2$

$Ix = 16000 cm^4 = 16000 \times 10^4 mm^4$

$\delta = \dfrac{5 \times 100.83 \times 10^3 N \times (12990)4mm^4}{384 \times 12.99 \times 10^3 mm \times 205 N/mm2 \times 16000 \times 104mm4}$

$= \dfrac{1.4355 \times 10^{22} mm}{1.6361 \times 10^{26}}$

$= 8.77 \times 10^{-5} mm: OK$

But

$\delta < L/360$

$8.77 \times 10^{-5} mm < 12990mm/360$

$8.77 \times 10^{-5} mm < 36.08mm: OK$
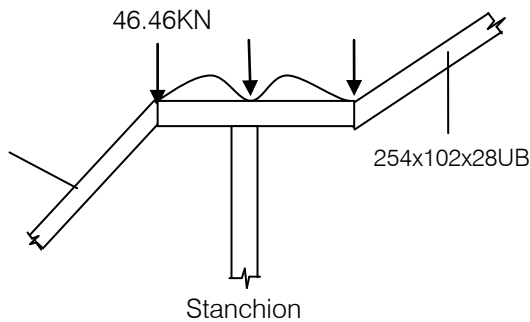
Fig. 1: Moment on the Stanchion

Bending moment

$BMmax = 46.46 \times 1.2 + 12.22 \times 1.2/2$

$= 63.08KNm$

Adopt 254x102x28UB since 77.43KNm > 63.08KNm

Column

Dead load, gk =

$46.46 + \dfrac{6.64}{2} + \dfrac{12.22}{2} + \dfrac{13kg/m \times 10 \times 10^{-3}}{2} + 1.78$

$= 57.74KN$

Live load, qk $= \dfrac{12.99 \times 1.2 \times 4.48}{2}$

$= 37.80KN$

Design load, n $= 1.4(57.74) + 1.6(37.80)$

$= 141.32KN$
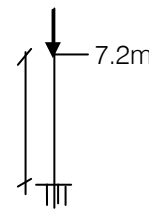
Effective length

$$Le = 1.0L \qquad [25]$$

Where, L = 7200mm

Therefore, Le = 1.0 * 7200 = 7200mm

Assume section 203*203*46UC

T = 11.0mm; area of section, $Ag = 58.7 cm^2$

Radius of gyration, Vy = 5.13cm

b/T = 9.25; d/t = 22.3

Since T ≤ 16mm $Py = 275 N/mm^2$

For outstand b/T = 15Ɛ semi compact

For web d/t ≤ 40Ɛ

$Ɛ = (275/Py)^{1/2} = (275/275)^{1/2} = 1$

b/T $= 9.25 ≤ 15Ɛ$

$= 9.25 ≤ 15 \times 1$

$= 9.25 ≤ 15$

d/t $= 22.3 ≤ 40Ɛ$

$= 22.3 ≤ 40 \times 1$

$= 22.3 ≤ 40$

$$PC = Agpc \qquad [26]$$

$ʎ = LE/Vy = 7200/(5.13 \times 10) = 140$

Using curve C; assuming it buckle along y-y with S275

$Pc = 76 N/mm^2$

But $Ag = 58.7 cm^2 = (58.7 \times 10^2)mm^2$

$PC = Agpc$ (from eq.26)

$= (58.7 \times 10^2)$ mm^2 x 76N/mm^2

$=446120N \times 10^{-3}$

$=446.12KN > 141.32KN$: OK

Foundation

Axial Load = 141.32KN

Stanchion load from 203*203*46UC

$=46.1kg/m *10* 0^{-3}=0.461KN$

Allowable bearing capacity = 233KN/m^2

Assume footing weight = 50KN

Fcu=25Nmm^2; Fy =460N/mm^2

Concrete cover =50mm

Live load

$=4.85(2.4 \times 7.67)+(1.2 \times 7.67/2) \, 4.85$

$=111.59KN$

Dead load = $141.32+0.461+50=191.78KN$

Serviceability limit state

$=1.0gk+1.0qk$

$=191.78+111.59=303.37KN$

Required base area

Area of footing = $\dfrac{\text{Service load}}{\text{Bearing capacity}}$

$=\dfrac{303.37KN}{233KN/m2}$

$=1.302m2$

This provides 1300 mm x 1300 mm x 450 mm footing

Ultimate Limit State

$n = 1.4gk+1.6qk$

$= 1.4 \times 141.78+1.6 \times 111.59$

$= 377.036KN$

Earth pressure $=\dfrac{377.036KN}{1.69m^2}$

$= 223.09KN/m^2$

Assume 450mm Thick Footing

Concrete cover 50mm

Assume 20mm ø bar in both direction

Then

d=h-c- ø /2=450-50-20/2=390mm

Punching Shear

Critical perimeter =col perimeter+4$\prod$d [27]

$=4 \times 203+4 \times 3.142 \times 390$

$= 5712mm$

Area within Perimeter

$= (203+4d)^2-(4-\prod)(2d)^2$

$= (203+4 \times 390)^2-(4-3.142)(2 \times 390)^2$

$= 2.58 \times 10^6 mm^2$

Punching shear force

$VED=456(1.692-2.58)=125.9KN$

Punching shear force=VED/col perimeter x d

$=125.9 \times 103/5712 \times 390$

$= 0.056N/mm^2$

Bending Reinforcement

$M=223.09 \times 1.3 \times \dfrac{0.525^2}{2}$

$= 39.97KNm$

$= 0.00808 \leq 0.156$

For the Concrete

Mu=0.156fcubd^2

$= 0.156 \times 25 \times 1300 \times 390^2$

$= 771.15KNm > 39.97KNm$

K = M/ fcubd^2

$= \dfrac{39.97 \times 10^6}{25 \times 1300 \times 390^2}$

III. Conclusion

The outputs of the design analysis indicate that the chosen sections for all the structural members of the footbridge are adequate in term of ultimate and serviceability considerations. The soil analysis shows that it would be able to withstand the load from the columns and vibrations from vehicular movement.

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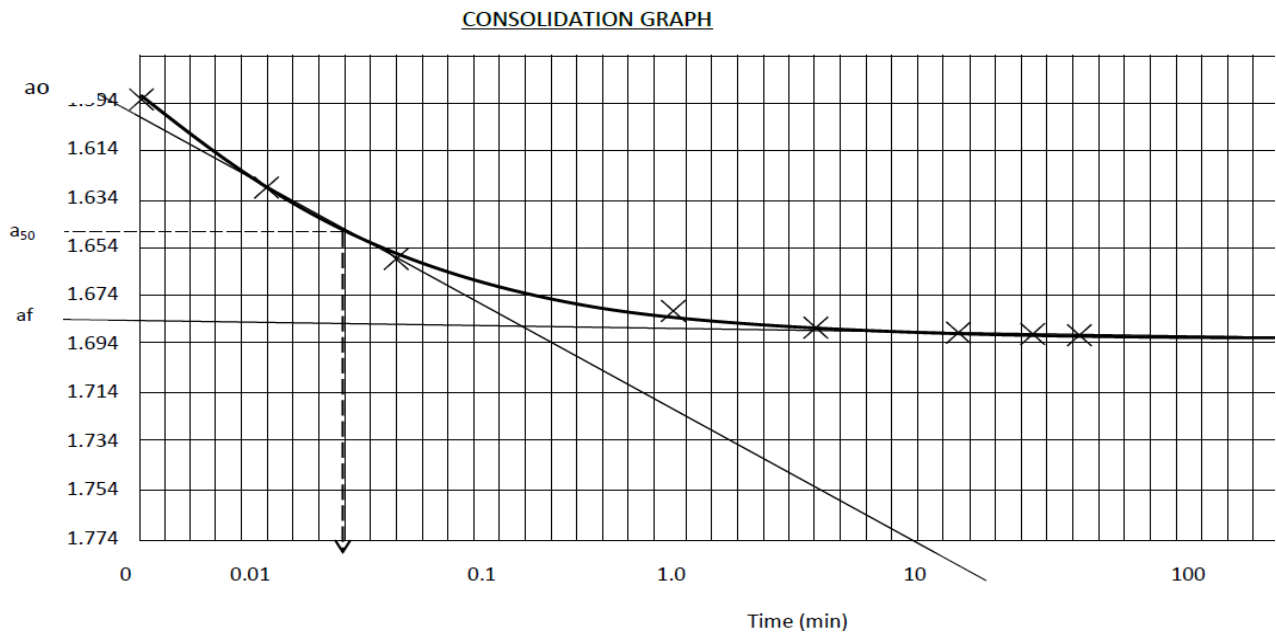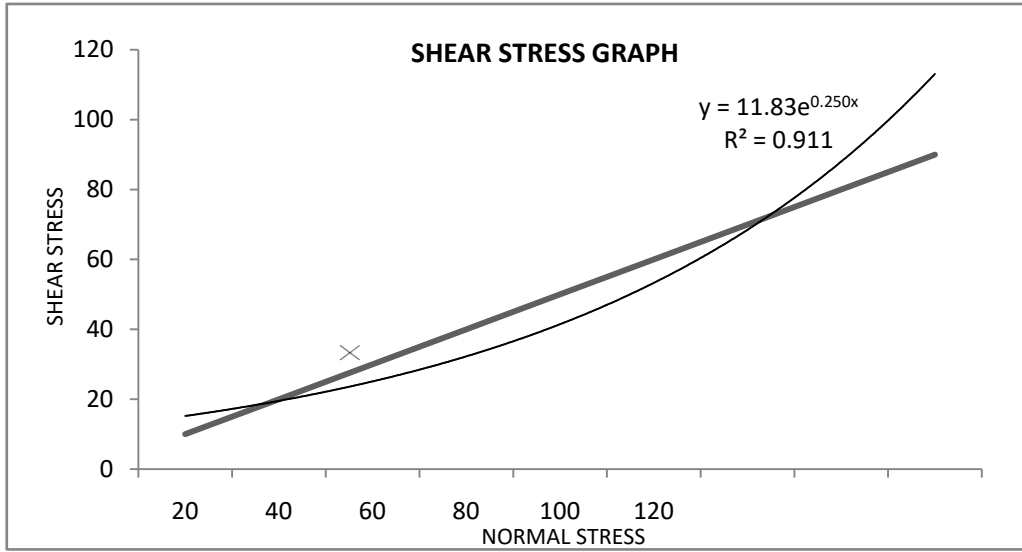
SHEAR STRESS GRAPH

$y = 11.83e^{0.250x}$

$R^2 = 0.911$

SHEAR STRESS

NORMAL STRESS

CONSOLIDATION GRAPH

ao

1.594

1.614

1.634

a_{50}

1.654

1.674

af

1.694

1.714

1.734

1.754

1.774

0   0.01   0.1   1.0   10   100

Time (min)

Experimental and Theoretical Studies of the Peculiarities of Concrete Behavior in Time and the Concrete Limit Characteristics from the Standpoint of Creep Adsorption Theory

By Merab Lordkipanidze, Olgha Giorgishvili, Iuri Salukvadze, Nika Botchorishvili & Aleksandre Tatanashvili

Georgian Technical University

Abstract- Based on the experimental and theoretical studies from the standpoint of the adsorption theory of creep, the authors show the peculiarities of the concrete behavior in time and propose a universal graph of the limit characteristics of concrete, including: the limits of strength, elastic deformation, linear creep, endurance.

It is established that the limit strength value R changes in time and depends on the velocity of load application, while the elastic deformation limit is a constant value and does not depend on the age of concrete and the load application velocity.

Keywords: concrete instant strength, over time strength of concrete, strain, creep, persistence limit, durability limit.

GJRE-E Classification: FOR Code: 290804

EXPERIMENTALANDTHEORETICALSTUDIESOFTHEPECULIARITIESOFCONCRETEBEHAVIORINTIMEANDTHECONCRETELIMITCHARACTERISTICSFROMTHESTANDPOINTOFCREEPADSORPTIONTHEORY

Strictly as per the compliance and regulations of:

Experimental and Theoretical Studies of the Peculiarities of Concrete Behavior in Time and the Concrete Limit Characteristics from the Standpoint of Creep Adsorption Theory

Merab Lordkipanidze [α], Olgha Giorgishvili [σ], Iuri Salukvadze [ρ], Nika Botchorishvili [ω]
& Aleksandre Tatanashvili [¥]

Abstract- Based on the experimental and theoretical studies from the standpoint of the adsorption theory of creep, the authors show the peculiarities of the concrete behavior in time and propose a universal graph of the limit characteristics of concrete, including: the limits of strength, elastic deformation, linear creep, endurance.

It is established that the limit strength value R changes in time and depends on the velocity of load application, while the elastic deformation limit is a constant value and does not depend on the age of concrete and the load application velocity.

The maximal limit deformation of concrete of any composition is a constant value under repeated loads as well as under the constant load.

The breaking of concrete in time occurs due to the combined action of external forces, which is expressed in terms of the trapezoid area, and due to an additional effect of the wedging action of water which is equal to the triangle area.

According to the theory on the nature of concrete creep, the cause of the creep in the region of elastic deformation is the influence of water adsorption, which shows itself in its wedging action on the reversible micro cracks of concrete, i.e. the creep in the elastic deformation region is completely reversible, and the wedging action of water can be regarded as an additional stress to the stress due to load.

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I. Introduction

According to the theory on the nature of concrete creep, the cause of the creep in the region of elastic deformation is the influence of water adsorption, which shows itself in its wedging action on the reversible micro cracks of concrete, i.e. the creep in the elastic deformation region is completely reversible, and the wedging action of water can be regarded as an additional stress to the stress due to load [1], [2].

The action of adsorbed layers of water reduces to their two-dimensional migration over the surfaces of micro cracks which are under the action of two-dimensional pressure in the mouths of further water

motion, thereby leading (in constant external conditions) to the increase of deformation. The effect of this pressure is equivalent to the increase of external force F by the value $\Delta F \approx \sigma_0 - \sigma_V$ which replaces the action of adsorbed layers which are its mechanical equivalent [3]. Quite a lot of works are devoted to the investigation of problems concerning the mechanical characteristics of concrete, see e.g. [1], [2], [3].

II. Materials and Methods

The results of the studies carried out in this connection and confirmed by experimental data are presented in Fig. 1 in the form of a theoretical graph.

A concrete prism is subjected to axial compression or tension and its deformations expressed in terms of the coordinates σ, ε with the origin at the point 0^1 are measured. After recording the moment of concrete deformation origination at the point 0, a minimal breaking load is instantly applied to the prism. We fix D and N. Readings are taken starting not from 0^1 but from 0 which is the real origin of the coordinates since concrete begins to work from this point. Connecting 0 with N we get the triangle 0DN which expresses the strength characteristic showing the capacity of concrete of every composition and age for work. 0D corresponds to the actual concrete ultimate strength R which is the maximal stress under the minimal, instantly breaking load instantly applied to the area of the working cross-section of the concrete element. Therefore R is the well-defined strength characteristic that fixes the increase of strength depending on a degree of the restraint of tensile deformation of concrete since no irreversible micro cracks might have developed in concrete by the moment of its instant breaking. The limit deformation $\varepsilon_{\text{пр}}$ corresponding to R has the following inherent peculiarity: $\varepsilon_{\text{пр}}$ is constant, not depending on the age of concrete. ON expresses the rectilinear line between the coordinates σ and ε the tangent of whose angle of slope to the abscissa axis is the concrete elasticity modulus.

These facts indicate the following: 1) the concrete works by Hooke's law until it reaches R and its

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elasticity modulus is a constant value not depending on a degree of concrete tension; 2) irreversible micro cracks appear and develop in concrete only after it reaches R; 3) in the presence of surface-active substances there is no weakening of the cohesion between particles takes place in the elastic deformation region; 4) the limit of concrete elastic deformation is R; 5) the breaking of concrete occurs in two ways: if the creep deformation is non-damping, then concrete breaks on reaching R and, simultaneously, it breaks for the corresponding $\varepsilon_{\text{пр}}$in the conditions of increased loading. Below we give the physical interpretation of these two cases of concrete failure.

Let us draw the vertical line through N normally to the abscissa axis. For the tested concrete with the every index, independently of the concrete age the vertical line, which with time grows up to N, limits all ultimate deformations irrespective of a loading mode (loading conditions).
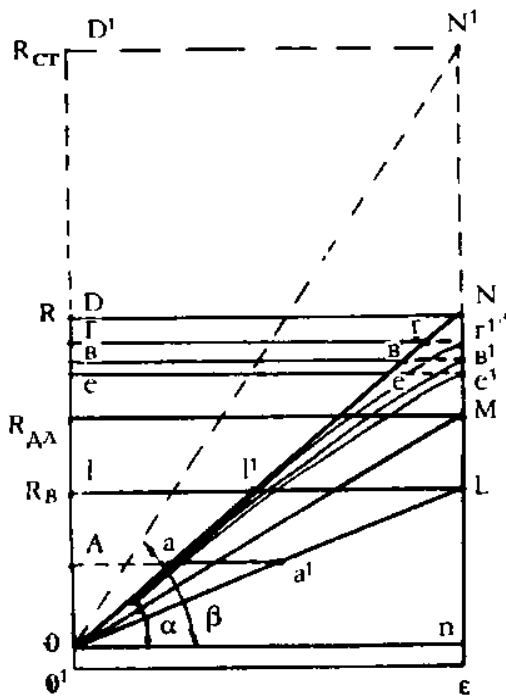
Fig. 1: Universal theoretical graph of the concrete limit characteristics

As different from the central tension of concrete, for axial compression we observe the restraint of tensile deformation caused by the friction of the concrete end faces against the cheeks of the press. Thus, if the friction is removed, the refraction points of the curves $0e^1$, $0b^1$ and $0r^1$ on σ, ε (Fig.1) of both concrete specimens will lie on the vertical line Nn and will simultaneously fix the end points (e^1,b^1,r^1) of maximal limit deformations and the moment of concrete breaking. They will lie higher with an increase of the velocity of concrete loading. If the loads corresponding to the refraction points e^1, b^1 and r^1 are instantly applied

to the concrete prisms, then on the straight line 0N we instantly obtain the points e, b and r which fix elastic limit deformations due to load application. However the breaking does not occur until the initial creep at these points reaches the corresponding points e^1,b^1,r^1 , i.e. until the total limit deformation reaches the vertical line Nn and becomes equal to the deformation DN and as elastic as the latter.

Thus the breaking of concrete occurs only after concrete reaches R and $\varepsilon_{\text{пр}}$. However this happens only in the case of non-damping creep of concrete, i.e. after the ultimate strength is reached, it is immediately followed by additional stress. All this indicates that in the elastic deformation region, the concrete works by Hooke's law. The curvilinear lines are obtained because not deformations due to loading are plotted on the abscissa axis but total deformations produced in particular by additional stress which is not plotted on the ordinate axis.

From the above-said it follows that creep deformation increases with time and at the level of the point M becomes maximally limiting. If above the point M the creep deformation was non-damping, at the point M it subsides and below the point decreases in direct proportion to the load by the straight line OM. At the level of the point M, simultaneously with achieving maximal limit creep deformation and therefore simultaneously with deformation damping, the strength of concrete stops growing in time and, with achieving R and $\varepsilon_{\text{пр}}$, because of the damped creep there will be no additional stress and concrete will not break. Therefore in our experiments the breaking of concrete occurs not with achieving R and $\varepsilon_{\text{пр}}$but after if the load increases.

The strength characteristic of concrete longevity at the considered point M of its stress-strained state is the limit of durable resistance of concrete, which is the limit stress under load and for which no breaking occurs and at the same time such factors are achieved as creep deformation damping, maximal limit value of creep deformation, termination of concrete strength growth in time, and the concrete actual limit strength R. However the difference $R_{\text{ст}}$-$R_{\text{дл}}$ is spent on the creep of concrete. Therefore the most important task of increasing the useful strength needed for maintaining the carrying capacity of concrete is to decrease the creep deformation of concrete.

The limit characteristic of concrete longevity is the fatigue (endurance) limit $R_{\text{в}}$ which is the maximal stress of concrete subjected to the action of repeated loads under which the creep gets damped. R and its corresponding $\varepsilon_{\text{пр}}$ are achieved, while the breaking does not occur. If by applying repeated loads to concrete of any age we quickly achieve the maximal ultimate deformation, but will continue to apply repeated loads, then, with time, the concrete age and strength/increase. In order to keep the maximal deformation constant since it tends to decrease it is necessary to increase the

repeated load until the moment at which the concrete strength stops to grow. In that case we achieve $R_{ст}$ and $R_{в}=R_{дл}$. Therefore, under both repeated and constant loads, the maximal limit deformation of concrete is a constant value and does not depend on the concrete age.

To conclude, it should be emphasized that the peculiarities of the work of concrete in time and its limit characteristics obtained theoretically (Figure 1) are completely confirmed by experimental data (Figure 2).

III. Experimental Confirmation of the Diagram of the Concrete State under Free Axial Compression and Tension

Proceeding from the principles of the adsorption theory of the nature of linear creep of rigid bodies, we performed experimental studies of the limit strength and deformation characteristics of concrete under free axial compression and central tension.

The concrete specimens were prisms of $10\times10\times40$ cm and cubes of $10\times10\times10$ cm. The consumption of materials per 1 kg//m^3 was: cement M4000 – 320, gravel – 1180, sand – 650, water – 180 (2330 kg//m^3), vibration duration was 20 s, humidity 90%, temperature 20^0C.

The molds were removed from the specimens two days after manufacturing and then stored in the test room with normal thermal conditions.

We described in detail only the experiments with the specimens of three-month age since the specimens of 9 and 16 months of age were tested analogously (Figure 2).

The $10\times10\times40$ cm concrete prisms were tested for axial compression on the press H-50. Friction between the end faces of the prism and the plate of the press was removed by applying paraffin to the prism end faces.

Longitudinal deformations were measured by resistance sensors with 50 mm base which were glued to the middle part of two opposite faces of the prisms. Readings of the sensors were recorded by two instruments with a scale division 10^{-5}.

The concrete was found to develop no deformation until the application of a certain amount of load (300 H in the considered case). This value was taken as the real origin (0) of the coordinates ρ, ε (Figure 2,a). Further, an instant maximal breaking load was applied with a velocity of 15.0 MPa/s. At the time of specimen breaking, we simultaneously fixed the breaking load by manometer readings, and the limit deformation by two measuring instruments.

OD, the value of the real strength limit R=25.0 MPa was plotted on the ordinate axis, while DN, the value of limit deformation (shortening) of concrete $\varepsilon=104\times10^{-5}$ was plotted on the horizontal line. The vertical line Nn was drawn from the point D to the intersection with the abscissa axis. The point N was connected with the real origin 0. In this manner, using experimental data obtained by testing only one concrete specimen, we estimated the real strength limit, the corresponding limit elastic deformation $\varepsilon_{пр}$ and the straight line ON showing the relationship between concrete stresses and deformations and the tangent of whose angle of slope to the abscissa axis is the elasticity modulus of concrete. It is also important to note the obtained area of the triangle ODN represented the behavior of concrete at the time of its breaking.

Next we determined in a usual manner the limit strength and the limit deformation, i.e. at the load application velocity of 0.2 MPa/s. We constructed the stress-strain curve. As a result the strength limit was found to be equal to R'=21.6 MPa and the limit compression to $\varepsilon=108$x10^{-5}. It is specific to note that the point of refraction of the curve Γ' at the moment of breaking happened to lie on the vertical line Nn. Next we tested the experimental twin specimen to which we applied the breaking load R'=21.6 MPa. In this case the limit elastic deformation was $\varepsilon=90\times10^{-5}$, whose end point turned out to lie on the straight line of the elasticity modulus of concrete $\varepsilon=90$x$10^{-5.}$. Further, the process of deformation continued in time and its end point Γ' reached the vertical line Nn at the moment of breaking

(a)

(b)

(c)

(d)
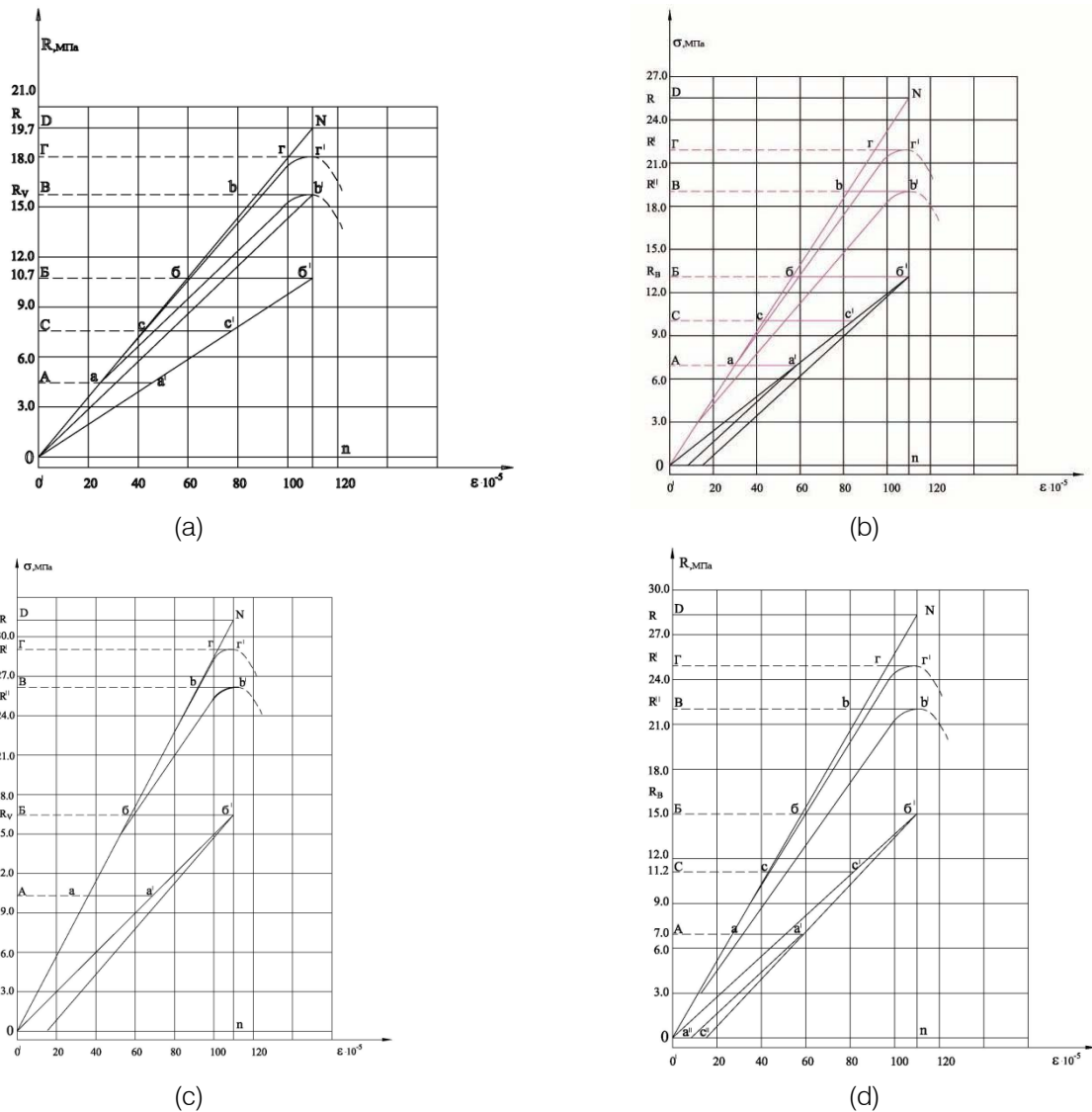
Fig. 2: Experimental justification of the universal graph of compression of concrete at the age of 1 month (a),
3 months (b), 9 months (c) and 16 months (d)

For a more complete understanding of the nature of limit characteristics of concrete, an experiment was run with the application of compressive breaking load at a much slower velocity, namely v=0.005 MPa/s and the stress-strain curve was constructed. The limit strength was R"=19.6 Mpa, while the end point of limit deformation ($\varepsilon=110 \times 10^{-5}$) coincided with the refraction point of the diagram and turned out also to lie on the vertical line Nn, which testifies to the fact that its value is equal to the instant maximal elastic deformation. When the instantly breaking load was applied to R"=19.6 the end point of elastic deformation reached the the elasticity modulus line, while the total deformation (elastic deformation in time) reached the vertical line Nn and coincided with the refraction point of this curve.

The curves OBbb', OCcc', Oaaa shown in Fig. 2 denote the limit of the fatigue (endurance) behavior of concrete under repeated static loads.

The theoretical principles of the universal graph of the limit characteristics of concrete and the peculiarities of its work were fully confirmed in the case of central tension as well (Figure 3).
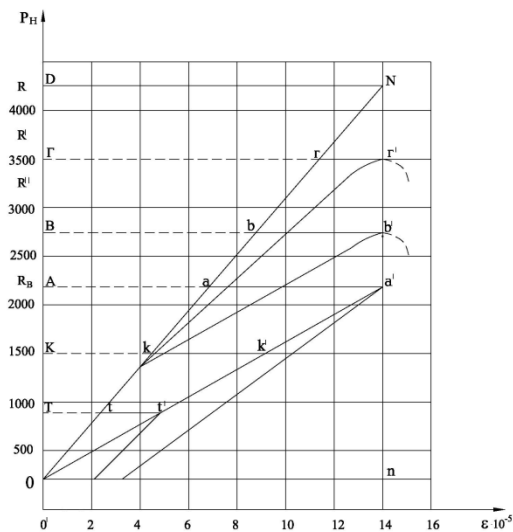
Fig. 3: Experimental justification of the universal graph for the central tension of concrete

IV. Conclusion

The analysis of the above graphs allows us to make the following conclusions:

1) The limits of structural changes of concrete are the real strength limit which is the maximal stress obtained by dividing the instantly applied load by the area of the working cross-section of a concrete element and the corresponding limit elastic deformation.

2) The concrete strength (R) changes with time and depends on the velocity of load application, while the limit deformation $\varepsilon_{\text{пред}}$, being wholly elastic, has its inherent peculiarity: for the concrete of any composition and any degree of its restraint the value $\varepsilon_{\text{пред}}$ is constant and does not depend on the age of concrete and the velocity of load application.

3) The law of concrete of concrete strength change in time is of the same character as the law of an instant change of the concrete modulus of elasticity since $\varepsilon_{\text{пр}}$ =const and R=εF, which is confirmed by the experiment.

4) Any failure occurs when concrete achieves the real strength limit and the real elastic deformation $\varepsilon_{\text{пр}}$. In this connection concrete performs the work equal to the area of the triangle 0Nn in Figure 2. With the decrease of the load application velocity the concrete strength R' decreases too. Experiments with dry, air-dried and water-saturated concrete showed that the additional stress is produced by the wedging action of water or, which is the same, by sorption load. In [4] it is stated that for calcium hydro silicates and Portland cement materials the sorption load of any value, including the maximal one, acts like mechanical load. Therefore the failure of concrete in time occurs under the total work of the external force which is expressed as the area of the trapezoid OB b'n, and also under the additional work of the wedging action of water equal to the area of the triangle BB'N, the combination of these both factors being equal to the area of the rectangle 0Nn.

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Mitigation of H2S Emissions by Recycling Discarded Gypsum Wall Boards in CLSM

By T Raghavendra, Y H Siddanagouda, Fayaz Jawad, C Y Adarsha
& B C Udayashankar

Visvesvaraya Technological University

Abstract- This paper highlights the benefits of incorporating wastes such as powdered gypsum wall boards (PGP) or drywalls, ground granulated blast furnace slag (GGBS) and quarry dust on improved performance of Controlled low strength materials (CLSM), which is a self-flowing cementitious backfill material. Drywalls, a construction & demolition waste, are known to pollute atmosphere by releasing harmful H2S gas when dumped at landfills. In literature, ternary binder combination of powdered gypsum wall boards, fly ash and cement resulted in reduced compressive strength values of CLSM specimens at 56 days when compared with 28 days. This paper investigates fresh and hardened properties of novel CLSM mixtures, and emphasizes on the incorporation of GGBS instead of fly ash which is efficient and helps to overcome the reduced compressive strengths at later ages. However, this was observed to be more effective only at lesser water contents.

Keywords: H2S; gypsum wallboard; ggbs; quarry dust; recycle.

GJRE-E Classification: FOR Code: 290899

MITIGATIONOFH2SEMISSIONSBYRECYCLINGDISCARDEDGYPSUMWALLBOARDSINCLSM

Strictly as per the compliance and regulations of:

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Keywords: H_2S; gypsum wallboard; ggbs; quarry dust; recycle.

I. Introduction

Construction and Demolition (C & D) wastes are generated in large quantities due to increase in construction activities such as construction of new buildings, demolition of old and obsolete buildings, renovations of existing buildings, etc; and gypsum wall boards are contributing in huge numbers to these wastes as they are most commonly used construction material for interior works. About fifty percent of the C & D wastes are dumped at landfills and remaining are being recycled [1]. These wastes from construction activities are usually re-used as recycled concrete aggregates [2]. The waste drywall pieces are thrown away and piled as debris near the construction sites and later local public waste disposal vehicles transport them to nearby landfills. It is reported that certain sulphur reducing bacteria's react with these gypsum wall board wastes; and as these wastes are composed of calcium and sulphate, with availability of necessary temperature, moisture and anaerobic conditions at landfills, hazardous H_2S (hydrogen sulphide) gas is released to atmosphere [3–7]. From public health point of view and also increasing public interest litigations necessitates that these wastes should be recycled in large numbers instead of being dumped at landfills. In literature [7], an attempt was made on possible re-use of wasted drywalls in concrete. It was concluded that about 60% (by weight) of total binder i.e. cement may be replaced by the combination of Class C fly ash and gypsum wall boards, and only 10% (by weight) of the total cement content could be replaced by powdered gypsum wall board. Cement replacement of 10% (by weight) will not help for large scale re-use of these gypsum wall board wastes. Hence an alternative should be encouraged possible re-use of these gypsum wall board wastes in large quantities for sustainable development of concrete industry.

Controlled low-strength material (CLSM) is an obvious choice for re-use of many types of waste materials such as GGBS, fly ash, C & D wastes, etc in large quantities [8–10]. Hence an attempt was made for possible re-use of gypsum wall board wastes in CLSM. Fly ash is a coal combustion product having fine particles which are responsible for pollution of atmosphere and environment as a whole resulting in disturbed ecological cycles and hazardous environment, and it has been popularly used, as a replacement to binder, in cement [11,12], CLSM [13], geomaterials [14] and concrete [15,16]. Quarry dust is a waste from stone industry collected out of many processes involved in the manufacture of final product such as overburden, screening, sludge and fragments. Seventy eight to fifteen percent of total quarried material is collected as quarry dust [17,18] and this waste is being used as a replacement to aggregates in CLSM and concrete [19–21]. Ground granulated blast furnace slag (GGBS) is an industrial by-product and its use in concrete industry is recognized by Leadership in Energy and Environmental Design (LEED). Hence use of GGBS in CLSM will add points towards LEED certification and improve the sustainability of the project. GGBS is widely used as secondary cementitious material in CLSM [13,22,23], pozzolanic cements [24] and concretes [25].

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In this paper, CLSM mixes were proportioned for 1(binder):1(fine aggregate) ratio. Powdered gypsum wall board + Ground granulated blast furnace slag (GGBS) + cement; comprised the binder. Additional mixtures were produced with soda ash as an activator, to activate the ternary binder blend, and the results were compared with those without soda ash. Quarry dust fines were used in total replacement to natural river sand. About seventy four to eighty seven percent by weight of total binder content was replaced by binary blend of gypsum wall board + GGBS, and sixty one to fifty two percent by weight of total binder content was replaced by powdered drywall wastes. Soda ash if used as an activator has no appreciable increase in strength values. In literature [26] Class F fly ash was used and the CLSM mixes were named as GF series; it was reported that the 56 days strength reduced when compared to 28 days strength values, due to use of gypsum wall boards. However, in the present research work, GGBS was used instead of Class F fly ash; and it was found that the reduced compressive strengths was not observed at 56 days age for GGBS based CLSM mixes at 45% (by weight) water contents only[23]. This paper investigates the engineering properties of sustainable CLSM mixtures with generation of strength and flow phenomenological models [22], for possible re-use of these wastes as binders (fly ash or GGBS + powdered drywalls) and fine aggregates (quarry dust), in a total of twenty mixture combinations. Although the materials described in this paper, the flow and strength results discussed, are the same which are described in the literatures [23,26]; the density and settlement results of GG series are investigated and reported to facilitate

practical applications. However, an attempt has been made in this paper to comprehensively review the results as well as the phenomenological models which are described in the literatures [23,26].

II. Materials And Methods

a) Experimental Investigation

Twenty CLSM mix proportions were generated. Strength, density, settlement and flow properties were analyzed and assessed. CLSM mortar mixes using ground granulated blast furnace slag (GGBS) and powdered gypsum wall boards (PGP) as secondary cementitious material and quarry dust as fine aggregates, were termed as GG series. First, trial mixtures were produced to determine variations of water content, by measuring spread flow diameter and thereby calculate RFA values.

Table 1 gives the mixture proportions for GG series with respect to GGBS/C [Ground granulated blast furnace slag (GGBS) to cement (C)] and PGP/C [Powdered gypsum wall boards (PGP) to cement (C)] ratio variations, respectively. Water content for GG series were varied from 45% to 60% so as to get desired flow values in terms of RFA ranging from 5-15 [13], which is required for self-flowing and self-leveling consistency of the mix. For each series 80 specimens were cast i.e. 20 specimens were cast for each B/w (binder/water) ratio; five specimens each were tested at the increasing age of 3, 7, 28 and 56 days respectively. A total of 400 specimens were cast and tested for GG Series.

Table 1: Composition of CLSM mixtures – GG series

Mixture Name	GGBS/C ratio	PGP/C ratio	Cement (C), g/100g	Powdered Gypsum Wall Board (PGP), g/100g	Ground Granulated Blast Furnace Slag (GGBS), g/100g	Quarry Dust, g/100g
GG1	2	4.67	6.52	30.43	13.05	50
GG2	1.5	3.5	8.33	29.17	12.50	
GG3	1.2	2.8	10.00	28.00	12.00	
GG4	1	2.33	11.54	26.92	11.54	
GG5	0.86	2	12.96	25.93	11.11	

Note:

1. Mixtures containing cement, GGBS, powdered drywalls and quarry dust are termed as GG series. Variations of water contents was 45%, 50%, 55% and 60% (by weight), same as GF series (Raghavendra and Udayashankar, 2015).

b) Materials used and items of investigation

1) Binders
   i. Cement (C)
   ii. Ground granulated blast furnace slag (GGBS)
   iii. Powdered Gypsum Wall board (PGP)

2) Fine Aggregates:
   i. Quarry dust

3) Type of Curing:
   i. Air cured at standard room temperature ≈24°C

4) Tests Conducted
   i. Fresh state:
   ii. Spread Flow test and Marsh cone flow test and density.
   iii. Hardened state:
   iv. Unconfined Compression test, density and settlement tests on cylindrical specimens.

5) Size of Specimen
   i. Diameter = 40 mm; Height= 80 mm

6) Variable Parameters
   i. Cement, Class F fly ash and quarry dust, 1:1 mixtures
   ii. Cement, GGBS and quarry dust, 1:1 mixtures
   iii. GGBS to Cement ratio (GGBS/C): 2, 1.5, 1.2, 1 and 0.86
   iv. Powdered gypsum wall boards to cement ratio (PGP/C): 4.67, 3.5, 2.8, 2.33 and 2
   v. Water content (w %): 45%, 50%, 55% and 60%
   vi. Compressive strength, settlement and density tests: 3, 7, 28 and 56 days.

c) Material Properties

The materials adopted in this research are the same which are described in the literatures [23,26].

Ordinary Portland cement (C) of 53 grade was used and its physical properties were determined according to IS: 12269 [27] specifications. Initial and final setting times of cement were found to be 43 min and 218 min, respectively with a specific gravity of 3.09. Ground granulated blast furnace slag (GGBS) was used as secondary cementitious material and was procured from JSW Steel Ltd., at Toranagallu, Bellary - Hospet, Karnataka, India; having a specific gravity of 2.82. Waste gypsum wall board sheets were used as secondary cementitious material which were sourced from new construction sites and demolition sites in Bangalore, and was crushed manually. Powdered gypsum wall board passing through 4.75 mm sieve size was used with a specific gravity of 1.76. The specific surface area determined by Blaine's permeability method for cement, GGBS, powdered gypsum wall board and stone dust were 307m^2/kg, 327m^2/kg, 169m^2/kg and 381m^2/kg, respectively. Stone dust having a specific gravity of 2.46 was sourced from stone quarry waste dump site at Bidadi, Bangalore, Karnataka, India. Table 2 gives the elemental compositions of cement, powdered gypsum wall board, GGBS and quarry dust, obtained from the Scanning Electron Microscopy (SEM)–Energy Dispersive X-ray Spectroscopy (EDS). The particle size distribution of materials is illustrated in Fig. 1.
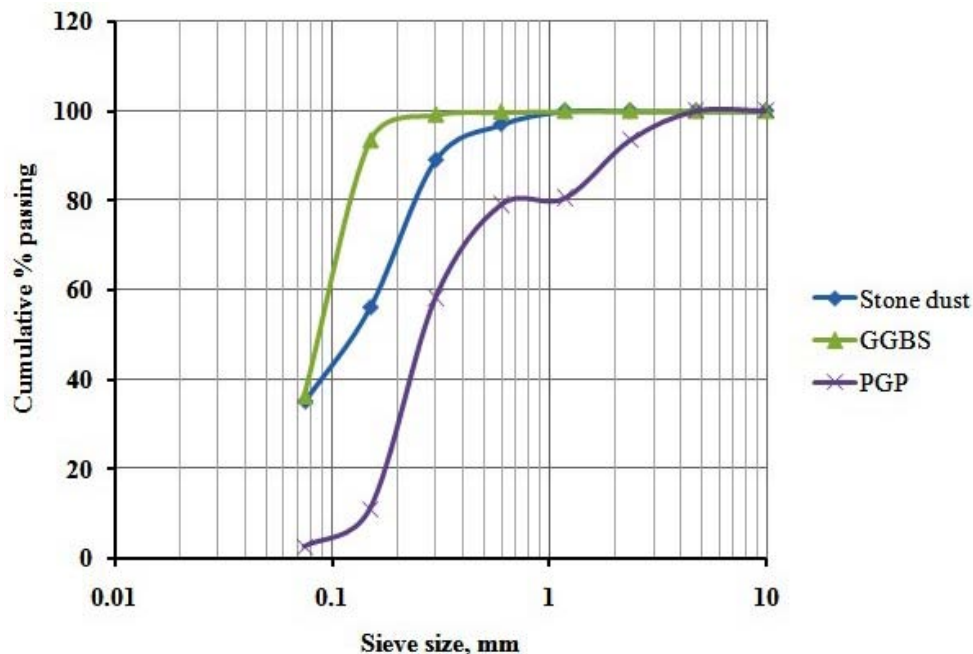
Fig. 1: Particle size distribution of materials.

d) Engineering Properties of CLSM

The spread flow test [26,28] was conducted using an open ended cylinder of 75 mm diameter and 150 mm height, according to standard [29]. The flow diameter (D) measured in six directions was averaged and relative flow area (RFA) was calculated using the formula $(D/75)^2-1$. Marsh flow test [26] was conducted using a brass cone of 10mm smooth aperture diameter, according to standard [30]. The Marsh flow time was averaged out of three trials. Un-confined compression strength tests [26] were carried out at 3, 7, 28 and 56 days age, respectively; using a modified CBR

apparatus. The results from minimum of five specimens were recorded and averaged. The density of CLSM was measured at fresh state and at the increasing ages of 3, 7, 28 and 56 days. As soon as CLSM mix was poured into the acrylic moulds of 40mm diameter and 80mm height, the weight of the mix at the fresh state was recorded. Later the weight and volume of CLSM specimen was measured at increasing ages. Finally the density was calculated by dividing the weight of specimen with the volume of the hardened cylindrical specimen. Settlement of CLSM was measured by recording the reduction in the height of hardened CLSM specimens at increasing ages of 3, 7, 28 and 56 days. The heights of the hardened CLSM specimens were measured at increasing ages. The difference in the height of CLSM specimen at fresh state and height of specimen at increasing ages gives the settlement of CLSM. For each series and with any particular water content minimum of five cylinders were measured and averaged to calculate settlement in Millimeters.

e) Analytical Investigation

    i. *Phenomenological Model*

        To generate phenomenological model for flow evaluation, a RFA value at particular water content is identified as reference RFA, such that the RFA values ranged from 5-15 which is required for self-flowing and self-leveling consistency [13,22]. For GG and GF series [23,26] 55% water content is suitable water content whose RFA values are taken as reference values in the generation of flow models since the RFA values are not exceeding 15. In the development of flow model all flow values were normalized with respect to a reference flow value at w=55% (by weight). The normalized values were plotted and the trend line equation represents the flow model in terms of RFA. The validation of this model for an independent set of data is also examined. Development of Phenomenological model for strength prediction a reference value of suitable B/w ratio was identified. Normalized strength values were calculated. For GG and GF series [23,26] 1.1 is suitable B/w ratio whose strength values are taken as reference values in the generation of strength models. The linear models were obtained by plotting normalized strength values of all the selected series against B/w ratios.

        Fig. 2 and 3, gives the generalized flow (at w=55%) and strength (at B/w=1.1) models based on the results of GG1, GG2, GG3 and GG5 series only. The generalized flow model for GG series is "{RFA / (RFA@w=55%)} = 0.066w - 2.6". The generalized strength models for GG series at 3, 7, 28 and 56 days are "{S / (S@B/w=1.1)} = 1.93 (B/w) - 0.926"; "{S / (S@B/w=1.1)} = 1.338 (B/w) - 0.357"; "{S / (S@B/w=1.1)} = 1.806 (B/w) - 0.839"; "{S / (S@B/w=1.1)} = 2.569 (B/w) – 1.481"; respectively.

$y = 0.066x - 2.600$
$R^2 = 0.934$

$y = 0.068x - 2.731$
$R^2 = 0.979$
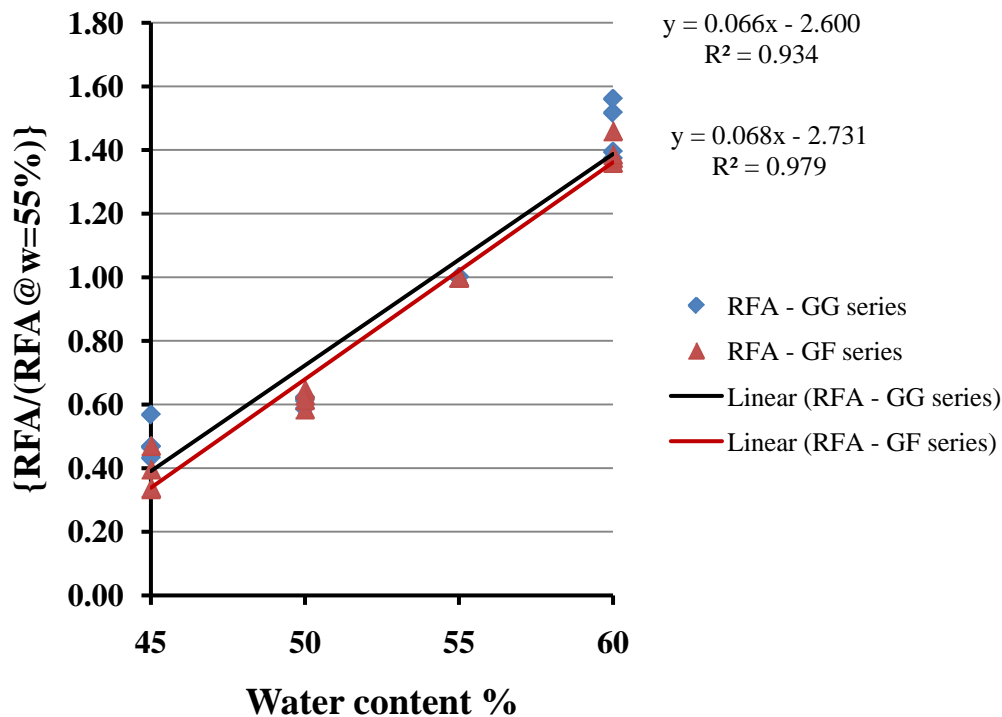
Fig. 2: Predictive flow models for GG and GF series.
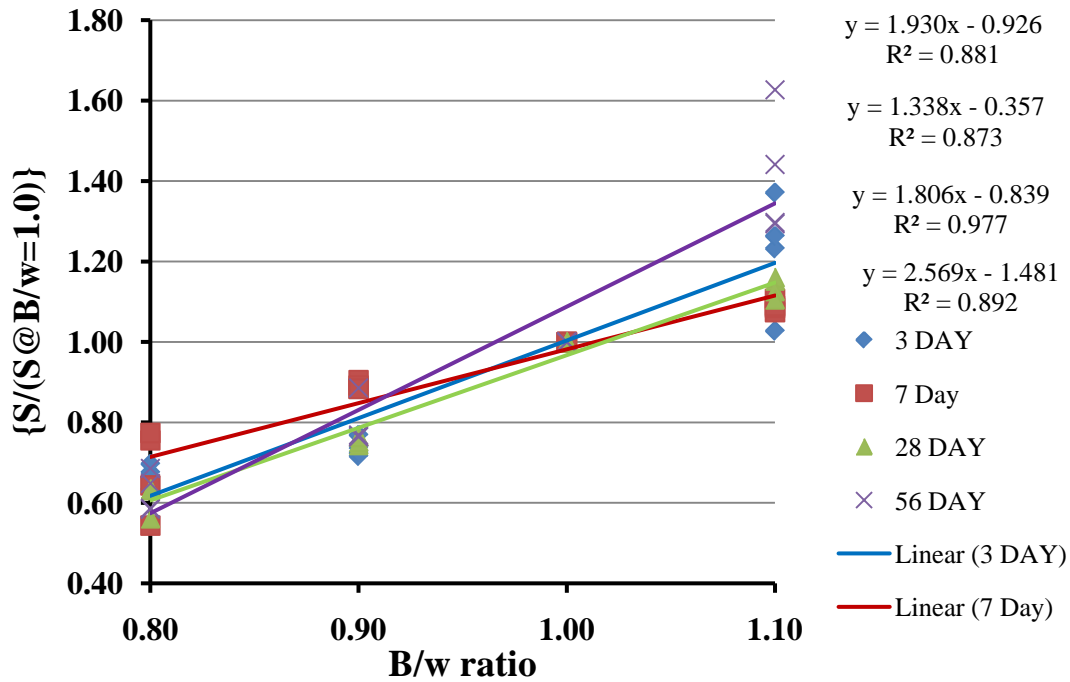
Fig. 3: Predictive strength models for GG series.

ii. *Comparison of predicted values by and experimental results*

The strength and flow predictive models [13,22] obtained for GG series are validated against GG4 series experimental values and are shown in Fig. 4 and 5.
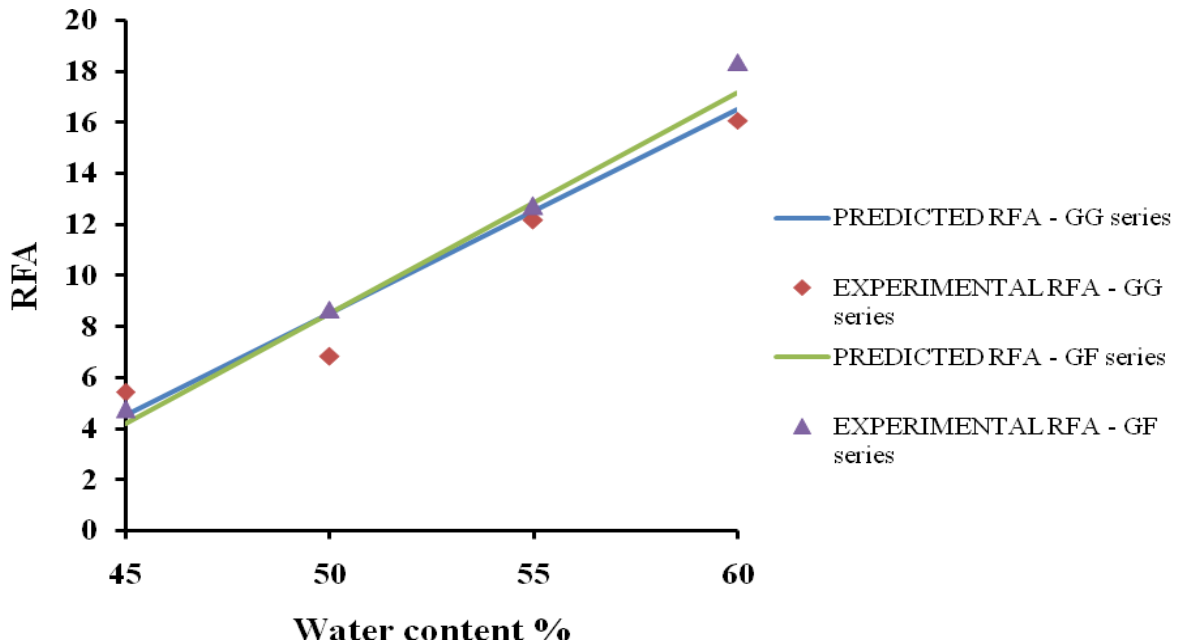
Fig. 4: Comparison of predictive and experimental flow - GG and GF series.
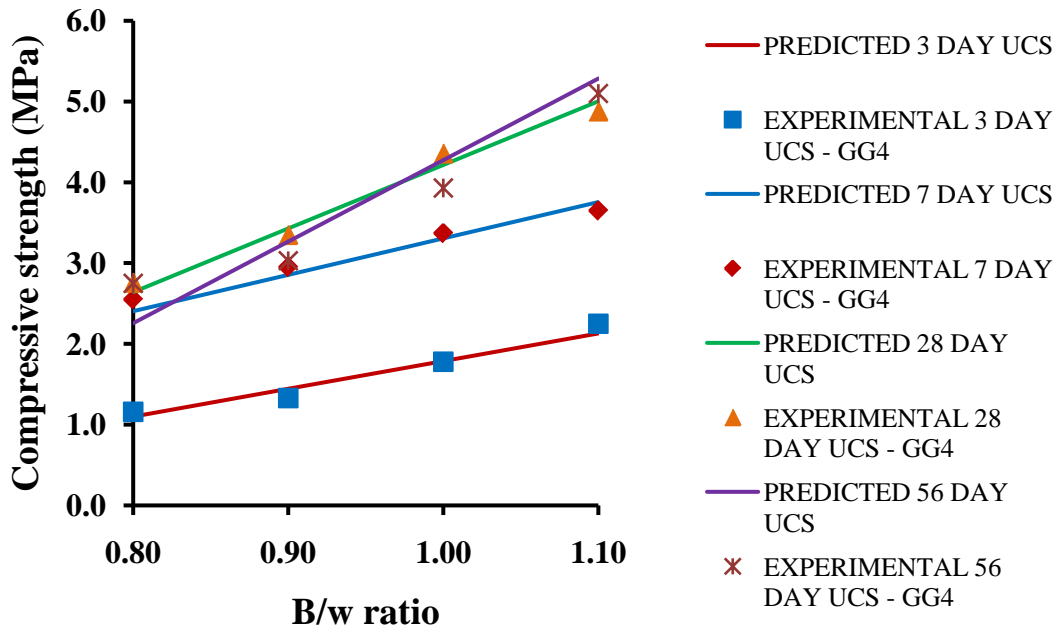
Fig. 5: Comparison of predictive and experimental strength values - GG series.

III. Results And Discussion

a) Flow and Strength

Average flow and strength results of GG series are given in Table 3, where the relative flow area (RFA) values are obtained from spread flow diameter as per equation (1). The RFA values of GF and GG series mixes ranged from 3.84 to 20.78 and 3.84 to 17.20, respectively. Almost all mixes have flow in the range of 5 – 15 RFA values are required for self-leveling and flowing consistency of the mixes [22]. Increase in water contents resulted in increased flow values. Higher water demand was recorded for GF and GG series CLSM mixes due to increase in cement and quarry dust contents which have large surface areas ($307m^2$/kg and $381m^2$/kg, respectively). Increase in water demand was observed for mixes with higher dosages of drywalls, GGBS and stone dust. This is due to high surface area and fine particle sizes of all the ingredients involved.

The Marsh flow time of GG and GF series mixtures ranged from 44 – 72 seconds and 39 - 110 seconds, respectively for water content of 60% (by weight of total mixture weight). Marsh flow time was zero seconds for all mixes with 45 - 55% water contents as flow did not occur at these water contents. Increased Marsh flow time was recorded for increased F/C and PGP/C ratios (by weight) due to the increased water demand with large contents of Class F fly ash and powdered drywalls, related to their fine particle sizes shown in figures of literature [26]. Comparing the spread flow and Marsh flow values it can be see that flow time increased with decreased flow diameter for CLSM mixtures at 60% water contents, hence a high drywall

content of sixty one percent by weight of binder is responsible for increased water demand when compared to other mixtures with lesser percentages of drywall contents i.e. about fifty two percent. Bleeding type of segregation was observed for all mixes with large water contents. CLSM mixes with zero Marsh flow was observed in CLSM mixes containing binder blend of cement + Class F fly ash + powdered drywalls [26]. Compared to GF series i.e. literature [26], GG series mixes have reduced Marsh flow time for higher dosages of drywalls, GGBS and stone dust. The fine particles of quarry dust with the largest specific surface area ($381m^2$/kg) contributed to the increased demand of water; particle size and physical characteristics are shown in Fig. 1 and Table 2.

Table 2: Elemental composition of materials, obtained from SEM – EDS analysis

| Spectrum processing: No peaks omitted |
| Processing option: All elements analyzed (Normalised) |
| Number of iterations = 4 |
| Standard: |
| C CaCO3 1-Jun-1999 12:00 AM |
| O SiO2 1-Jun-1999 12:00 AM |
| Mg MgO 1-Jun-1999 12:00 AM |
| Na Albite 1-Jun-1999 12:00 AM |
| Al Al2O3 1-Jun-1999 12:00 AM |
| Si SiO2 1-Jun-1999 12:00 AM |
| S FeS2 1-Jun-1999 12:00 AM |
| K MAD-10 Feldspar 1-Jun-1999 12:00 AM |
| Ca Wollastonite 1-Jun-1999 12:00 AM |
| Fe Fe 1-Jun-1999 12:00 AM |

Element	Cement		Powdered Gypsum Wallboard		GGBS		Quarry Dust	
	Weight %	Atomic %	Weight %	Atomic %	Weight %	Atomic %	Weight %	Atomic %
C K	5.40	9.62	---------	---------	4.42	7.99	---------	---------
O K	46.49	62.21	51.54	70.69	40.91	55.55	55.39	68.72
Mg K	---------	---------	---------	---------	4.06	3.63	---------	---------
Na K	---------	---------	---------	---------	---------	---------	2.35	2.03
Al K	2.61	2.07	0.00	0.00	8.17	6.58	5.54	4.08
Si K	7.64	5.82	0.44	0.34	14.00	10.83	32.82	23.19
S K	2.15	1.44	19.52	13.36	---------	---------	---------	---------
K K	0.93	0.51	---------	---------	---------	---------	3.19	1.62
Ca K	33.08	17.67	28.49	15.60	28.44	15.42	0.72	0.35
Fe K	1.70	0.65	---------	---------	---------	---------	---------	---------
Totals			100		100		100	

Table 3: Strength and flow results for CLSM mixtures of GG series

Mixture Name	w, %	B/w Ratio	Experiment results					
			Flow		Un-confined Compressive			
			RFA	Marsh time, Sec	3 Day	7 Day	28	56 Day
GG1 G/C=2; PGP/C=4.67	45	1.1	3.84	0	1.49	3.32	4.68	4.93
	50	1.0	5.08	0	1.45	3.09	4.06	3.03
	55	0.9	8.20	0	1.07	2.73	3.02	2.33
	60*	0.8	12.44	72	0.98	1.68	2.28	1.77
GG2 G/C=1.5; PGP/C=3.5	45	1.1	5.25	0	1.91	3.44	4.78	4.8
	50	1.0	5.76	0	1.55	3.17	4.12	3.33
	55	0.9	9.24	0	1.11	2.87	3.15	2.95
	60*	0.8	12.69	71	1.08	2.04	2.61	2.16
GG3 G/C=1.2; PGP/C=2.8	45	1.1	4.92	0	2.11	3.52	4.81	5.02
	50	1.0	6.47	0	1.67	3.23	4.27	3.88
	55	0.9	10.56	0	1.21	2.88	3.21	2.97
	60*	0.8	14.73	68	1.11	2.44	2.71	2.66
GG4 G/C=1; PGP/C=2.33	45	1.1	5.42	0	2.25	3.65	4.88	5.1
	50	1.0	6.84	0	1.78	3.37	4.36	3.93
	55	0.9	12.15	0	1.33	2.95	3.35	3.03
	60*	0.8	16.08	61	1.16	2.55	2.76	2.75
GG5 G/C=0.86; PGP/C=2	45	1.1	4.76	0	2.55	3.82	4.92	5.28
	50	1.0	6.47	0	1.86	3.45	4.45	4.07
	55	0.9	11.02	0	1.43	3.05	3.4	3.11
	60*	0.8	17.20	44	1.18	2.67	2.81	2.39

Note: * Bleeding was observed

The unconfined compressive strength results at 3, 7, 28 and 56 days for Class F fly ash based CLSM mixtures [26] ranged from 0.13 to 0.92 MPa, 0.35 to 2.10 MPa, 0.36 to 3.49 MPa and 0.26 to 2.53 MPa, respectively. The unconfined compressive strength results at 3, 7, 28 and 56 days for GGBS based CLSM mixtures ranged from 0.98 to 2.55 MPa, 1.68 to 3.92 MPa, 2.28 to 4.92 MPa and 1.77 to 5.28 MPa, respectively. The strength values are within the prescribed limits of 8.3 MPa [31] and most of the values are less than 2.1 MPa, hence suitable for applications requiring re-excavation. In Class F fly ash based mixes [26] 21% to 43%, 40% to 97% and 95.61% to 100% of the maximum strength is obtained at 3, 7 and 28 day age, respectively. While in GG series mixes 30.22% to 48.3%, 67.34% to 95% and 93.18% to 100% of the maximum strength is obtained at 3, 7 and 28 day age, respectively. CLSM mixtures with high drywall contents resulted in high early age strength development, owing to the significant presence of calcium, given in Table 2. It was observed that the strength increased at increasing ages of 3, 7 and 28 days for all the CLSM mixes of GF and GG series. Presence of sulfates in drywalls has a detrimental effect on strength values and resulted in reduction of strength values at 56 days age. In literature [26] about seven to thirty six percent strength reduction was noted at 56 days age when compared to that of 28 days age, for Class F fly ash based mixtures. The strength values reduced after 28 days i.e. at the age of 56 days, for GGBS based CLSM mixes of higher water contents of 50%, 55% and 60%. The comparison of the microstructure for GG1 series specimen powder, as shown in Fig. 6 and 7, clearly indicates the superiority of the lower water content mixtures (w=45%) in terms of closer bonding of ingredient materials with more formations of ettringite needles which is an indication of hydration activity, whereas, disintegration of ingredient materials with lesser or no indication of ettringites is observed in all higher water content mixtures. This reduced compressive strength values is due to the presence of sulfates in drywalls [23] used (refer Table 2) and their detrimental effects on hardened CLSM specimens leading to expansive cracks [7]. Similar behaviour was observed in GF series as reported in the literature [26] for CLSM mixes produced using the binder blend of cement + Class F fly ash + powdered gypsum wall board. Decrease in strength values were not observed for GGBS based CLSM mixes having the lowest water content of 45%. Hence it may be noted that binder blend of cement + GGBS + drywalls and lesser water contents, are effective in resisting the detrimental effects of sulfates present in drywalls. About 0.5-25% reduction in strength with respect to 28 day age strength was observed at 56 day age, in case of GGBS based CLSM mixtures. Compared to literature [26], GG series mixes

have high early age strength development and lesser percentages of strength reduction at 56 days age, due to the presence of sulfates in gypsum wall board. Marginal increase in strength values of GF and GG series CLSM mixes with soda ash may not be necessary for usual applications of CLSM. Strength values decreased with decrease in cement content and increase in water contents for both GG and GF series mixes. Considering increase in compressive strength values at increasing age for the mechanical evaluation on indication of pozzolanic activity, it may be observed that binder blend of drywall with Class F fly ash or GGBS resulted in pozzolanic activity with the cement hydration up to 28 days only and later reduction in cement hydration reduced pozzolanic activity and allowed sulphates present in drywall carry out detrimental effect leading to reduced strength values. In literature [7] concrete mixture made of ternary binder blend comprising of Class C fly ash, cement and drywalls was investigated. It was observed that this ternary blend had similar chemical compositions to that of another ternary binder blends comprising of Spray-dryer ash, Class C fly ash and cement; and clean coal ash, Class C fly ash and cement; which were adopted as blended cements and binder in concrete [12,16]. Microstructure, hydrated products and pozzolanic activities were also investigated [12,16]. According to literature [7] the concrete cylinders cracked and worst early age strengths were recorded due to excessive expansive reactions indicating the detrimental effects of sulfates present in drywalls.
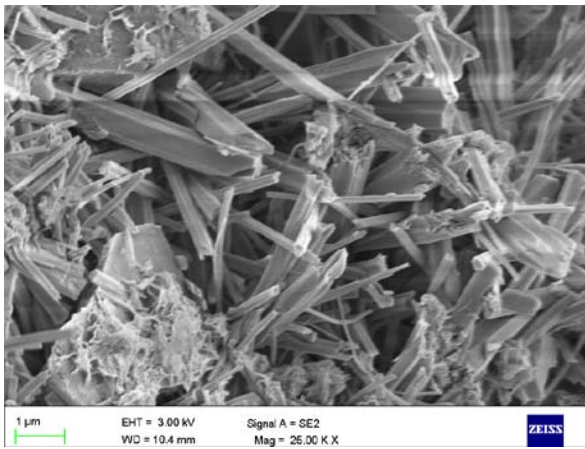
a)   w = 45%
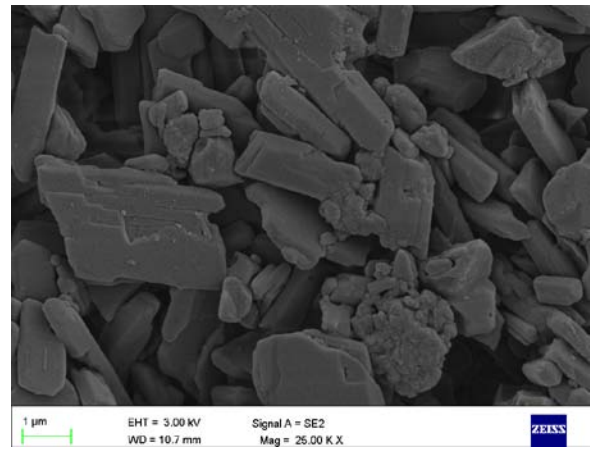
b)   w = 50%
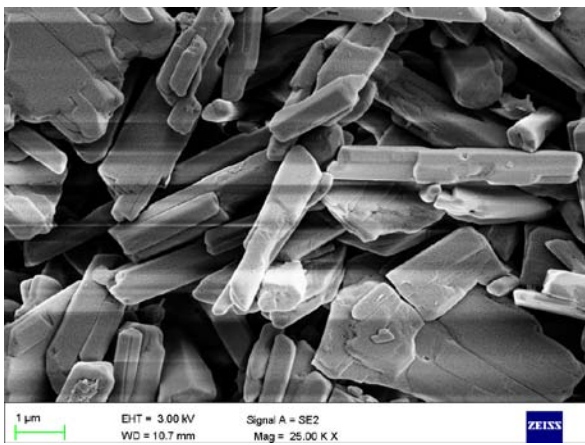
c)   w = 55%
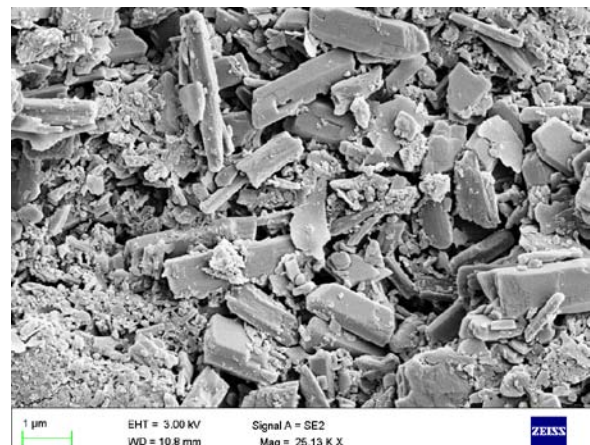
d)   w = 60%

Fig. 6: SEM images comparison of GG1 series at 28 days
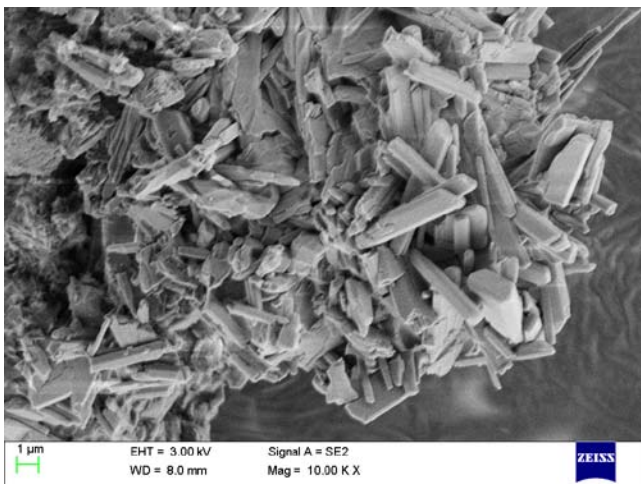
a)   w = 45%
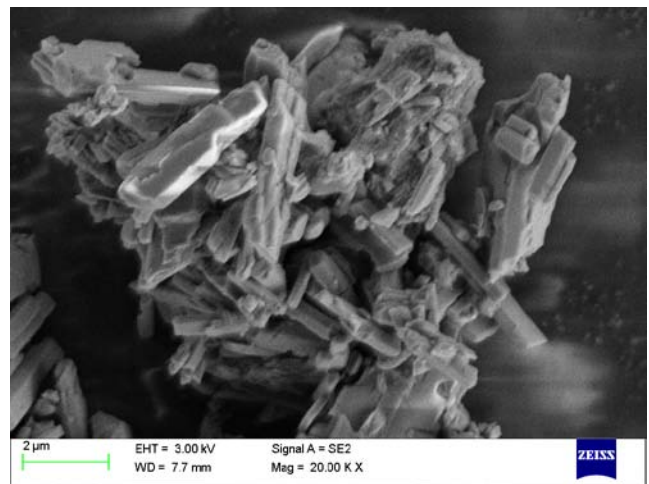
b)   w = 50%
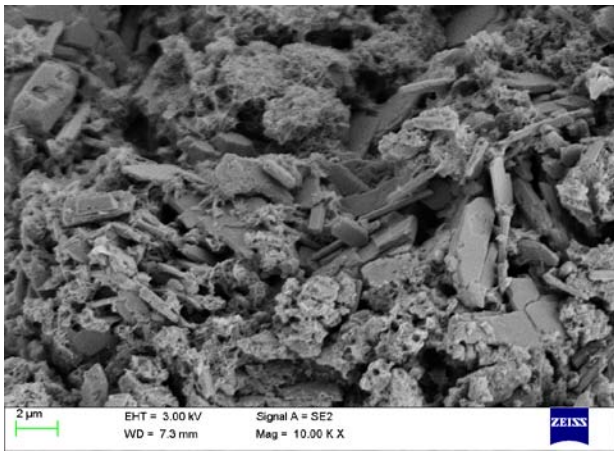
c)　w = 55%
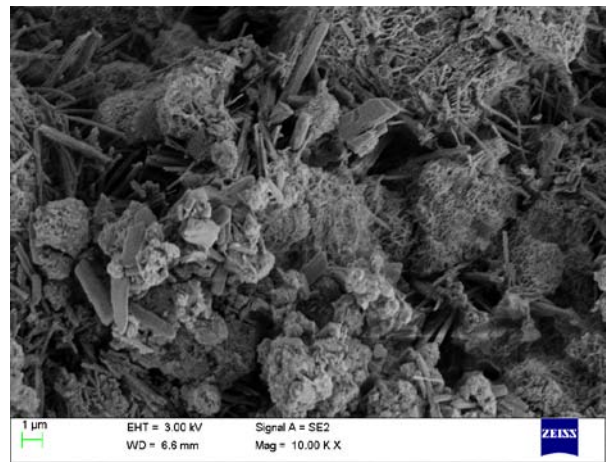
d)　w = 60%

Fig. 7: SEM images comparison of GG1 series at 56 days

b) *Settlement and Density*

GG series averaged settlement and density results are tabulated in Table 4 and 5, respectively. The settlement values ranged from 1 to 4 mm and 1 to 2 mm, for the GF and GG series CLSM mixes, respectively [26]. Early age settlement was observed and later ages it remained same. Settlement values are high when compared to that suggested in ACI-229R [31], since subsidence i.e. water and entrapped air being released while the CLSM mix specimen tries to consolidate itself; is not deducted from the final settlement readings. GF and GG series mixtures fresh density results ranged from 1611.44 to 1830.28 kg/m^3 and 1651.23 to 1800.44 kg/m^3, respectively [26]; which are less than and equal to the suggested range of 1840 to 2320 kg/m^3 [31] and better than most of the compacted earth fills. Hardened density results for Class F fly ash based CSLM mixes at

3, 7, 28 and 56 days ranged from 1495.65 to 1686.05 kg/m^3, 1336.49 to 1669.62 kg/m^3, 1154.91 to 1435.42 kg/m^3 and 1110.98 to 1337.20 kg/m^3, respectively [26]. Hardened density results for GGBS based CLSM mixes at 3, 7, 28 and 56 days ranged from 1550.74 to 1684.73 kg/m^3, 1224.27 to 1533.63 kg/m^3, 1207.95 to 1510.96 kg/m^3 and 1158.97 to 1420.31 kg/m^3, respectively. Majority of these results are within the suggested limits of 1440 to 1600 kg/m^3 for a CLSM mixture made of water, fly ash and cement [31]. Except for few mixtures high density was observed for GF3 series mixes and lower density for GF1 mixes. It was observed that in all the CLSM mixtures high density mixes did not result in high strength. The densities of GF and GG series mixes were almost in the same range and no significant difference was observed between both of these CLSM mixture combinations.

Table 4: Settlement results for CLSM mixtures of GG series

Mixture Name	w, %	B/w Ratio	Specimen Initial Height, mm	Settlement, mm			
				3 Day	7 Day	28 Day	56 Day
GG1	45	1.1	80	1.00	1.00	1.00	1.00
G/C=2;	50	1.0		1.00	1.00	1.00	1.00
PGP/C=4.67	55	0.9		1.00	1.00	1.00	1.00
	60	0.8		2.00	2.00	2.00	2.00
GG2	45	1.1		1.00	1.00	1.00	1.00
G/C=1.5;	50	1.0		1.00	1.00	1.00	1.00
PGP/C=3.5	55	0.9		1.00	1.00	1.00	1.00
	60	0.8		2.00	2.00	2.00	2.00
GG3	45	1.1		1.00	1.00	1.00	1.00
G/C=1.2;	50	1.0		1.00	1.00	1.00	1.00
PGP/C=2.8	55	0.9		1.00	1.00	1.00	1.00
	60	0.8		2.00	2.00	2.00	2.00
GG4	45	1.1		1.00	1.00	1.00	1.00
G/C=1;	50	1.0		1.00	1.00	1.00	1.00
PGP/C=2.33	55	0.9		2.00	2.00	2.00	2.00
	60	0.8		2.00	2.00	2.00	2.00
GG5	45	1.1		1.00	1.00	1.00	1.00
G/C=0.86;	50	1.0		2.00	2.00	2.00	2.00
PGP/C=2	55	0.9		2.00	2.00	2.00	2.00
	60	0.8		2.00	2.00	2.00	2.00

Table 5: Density results for CLSM mixtures of GG series

Mixture Name	w, %	B/w Ratio	Hardened Density, kg/m³				Fresh Density, kg/m³
			3 Day	7 Day	28 Day	56 Day	
GG1	45	1.1	1674.65	1473.19	1412.75	1380.01	1730.81
G/C=2;	50	1.0	1611.70	1440.45	1510.96	1352.31	1720.86
PGP/C=4.67	55	0.9	1573.92	1412.75	1392.10	1294.39	1700.97
	60	0.8	1597.67	1418.11	1352.82	1198.76	1661.18
GG2	45	1.1	1679.69	1533.63	1442.47	1345.77	1720.86
G/C=1.5;	50	1.0	1624.29	1475.71	1365.91	1228.92	1710.92
PGP/C=3.5	55	0.9	1619.75	1418.29	1363.90	1289.36	1700.97
	60	0.8	1591.55	1295.68	1263.04	1209.99	1661.18
GG3	45	1.1	1664.58	1518.72	1493.34	1420.31	1740.76
G/C=1.2;	50	1.0	1677.17	1420.31	1460.60	1334.69	1730.81
PGP/C=2.8	55	0.9	1631.84	1343.75	1329.05	1240.17	1671.13
	60	0.8	1603.79	1324.25	1307.93	1230.39	1651.23
GG4	45	1.1	1603.64	1404.19	1373.97	1337.20	1800.44
G/C=1;	50	1.0	1575.43	1444.48	1408.22	1323.61	1750.70
PGP/C=2.33	55	0.9	1587.47	1450.76	1387.50	1273.24	1720.86
	60	0.8	1581.35	1224.27	1207.95	1158.97	1651.23
GG5	45	1.1	1684.73	1410.23	1384.04	1313.53	1770.60
G/C=0.86;	50	1.0	1640.52	1450.76	1375.26	1328.84	1720.86
PGP/C=2	55	0.9	1567.06	1414.03	1340.57	1210.67	1710.92
	60	0.8	1550.74	1363.02	1291.60	1237.87	1681.07

c) Further Research

It is desirable to cast and test specimens at later ages to better understand the pozzolanic activity of the binder blend (GGBS + gypsum wallboard + cement) used, and efforts should be made to find out the possible new applications of this CLSM mix and wide spread use of predictive models and different material constituents involving more waste materials [26]. The results of such studies would directly benefit the society and environment protection initiatives.

IV. CONCLUSIONS

The below mentioned conclusions can be drawn based on the experimental results:

1) CLSM mixtures containing ternary binder blend of cement, ground granulated blast furnace slag and drywalls, reported reduced compressive strength values after 28 days age. However, the reduction in strength was not observed for mixes with water content of 45%. About -0.5 to -0.25% of strength reduction was observed for mixes with water contents of 50%, 55% and 60%, respectively. However, CLSM mixtures containing ternary binder blend of cement, drywalls and Class F fly ash, resulted in reduced compressive strengths at 56 days when compared to 28 days age at all water contents.

2) Use of GGBS instead of Class F fly ash along with cement and stone dust, is recommended for production of CLSM mixes with lesser water contents to effectively overcome the detrimental effects of sulfates present in drywalls. The lesser water content required may be determined based on the self-flow and consolidation criteria of a particular application.

3) Water demand of CLSM mixtures increased due to use of drywalls, GGBS and stone dust, in large quantities. Same was reported in GF series with use of Class F fly ash.

4) Spread flow and Marsh flow time for GGBS based CLSM mixes reduced when compared to Class F fly ash based mixes and Marsh flow time was recorded only for mixes of 60% water contents as other water contents resulted in zero flow.

5) Reduced GGBS/C and PGP/C ratios increased 3 days strengths, leading to high strengths up to 28 days age. Soda ash as an activator is not necessary for the production of CLSM mixtures containing drywalls. Same was reported in GF series with use of Class F fly ash.

6) Settlement of gypsum wall board CLSM mixes was observed more during early ages similar to conventional CLSM mixes.

7) Density results of drywall CLSM mixes were similar to that of conventional CLSM mixes.

8) GG4 and GF4 mixes flow and strength results were validated against the predicted values. The predictive models can be used for engineering applications. Same was reported in GF series with use of Class F fly ash.

9) Wasted drywalls re-use in CLSM will reduce pollution of atmosphere due to release of H_2S gas at landfills. Recycle of GGBS, drywalls and stone dust, which are by-products and waste materials, will reduce the burden on landfills and hence add to sustainability achievement of industries.

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Seismic Vulnerability Assessment of Adamson University Buildings' As-Built using Fragility Curves

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Abstract- Adamson University buildings' age ranges from 10 years to 86 years. The structures' age can be one of the factors of their vulnerability to seismic hazard. Possibility of having different damages after the event of seismic activities can be measured through structural modeling and subjecting the latter to earthquake simulation. In the field of structural reliability, fragility analysis can be used in the assessment of structures. This type of analysis can be carried out by taking the structural performance using nonlinear analyses: Pushover Analysis and Time History Analysis. Five (5) buildings in Adamson University were analyzed and modeled in a structural analysis software based from the developed as-built plans with rebound hammer test results for the compressive strength of concrete. These structural models were subjected through a total of 22 ground motion data (from both Philippine and Japan), with each ground motion data normalized from 0.1g to 1.0g of peak ground acceleration (PGA). The result of fragility analysis has some limitations such as the use of as-built plan of each building assessed. The lowest probability of exceedance of 9% at a PGA of 0.4g as per National Structural Code of the Philippines specification is based from the fragility curve of FRC Building under a "Complete Damage" or damage rank of "As".

Keywords: fragility curve, pushover analysis, time history analysis, probability ofexceedance.

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Keywords: fragility curve, pushover analysis, time history analysis, probability ofexceedance.

I. Introduction

In the webpage of the Philippine Institute of Civil Engineers, it was stated in one of its Fundamental Canons that: "Civil Engineers shall hold paramount the safety, health and welfare of the public and shall strive to comply with the principles of sustainable development in the performance of their duties (Philippine Institute of Civil Engineers Inc., 2018)." Hazards, vulnerability, and risks affect the safety of a structure and it is a challenge to civil engineers to assess those factors to existing and developing buildings. Furthermore, hazard is the possibility that a damaging phenomenon will likely to occur. Vulnerability is the area or extent that will be subjected to damages after an occurrence of a hazard. The expected losses from the occurrence of a hazard including the property damages, injuries, and loss of lives are the risks.

Nowak & Collins (2000) developed several methods in the analysis of structural reliability. These reliability methods are Monte Carlo Simulation (MCS), Latin Hypercube (LHC), First Order Reliability Method (FORM), Second Order Reliability Method (SORM), Hasofer-Lind Method of reliability index.

Requiso (2013) further enhanced his fragility curves by using nonlinear static analysis and nonlinear dynamic analysis. Two years after, Requiso's research was enhanced further the development of fragility curves by using the two aforementioned nonlinear analysis plus the application of interval analysis (Baylon, 2015). It is notable that the sophistication of these assessments increases the accuracy of the results. The studies of Baylon were followed by more seismic vulnerability assessments in the succeeding years.

One of the major fault lines in the Philippines is the Valley Fault System, which is composed of two sections: the 10km East Valley Fault and the 100km West Valley Fault. The West Valley Fault System (WVFS) traverses through a large portion of Metro Manila which could definitely endanger the lives of people, infrastructures, and buildings. For the past 1400 years, the West Valley Fault has moved 4 times and has a movement interval of 400 years. The last recorded movement of WVFS was on 1658 (Solidum Jr., 2015). The PHIVOLCS warned Metro Manila that the WVFS is ripe for generating a devastating magnitude of no less than 7.2 earthquake in the Richter scale or also called as "The Big One". According to PHIVOLCS, the said earthquake could be experienced in the near future. For that reason, the existing buildings must be assess for structural integrity (PHIVOLCS, JICA, & MMDA, 2004).

The buildings and facilities of Adamson University will be definitely affected if the "Big One" occurs. Using the PHIVOLCS' Fault Finder App, the nearest distance to the West Valley Fault System of Adamson University is 8.9 km as of 2013 mapped, as can be seen in Fig. 1.1 These buildings will be susceptible to such level of damage. As of this writing, a group senior civil engineering students conducted

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seismic assessment of Adamson University buildings with the use of "as-built" plans. The assessment from this study will help provide a seismic evaluation and assess the vulnerability of the structures.
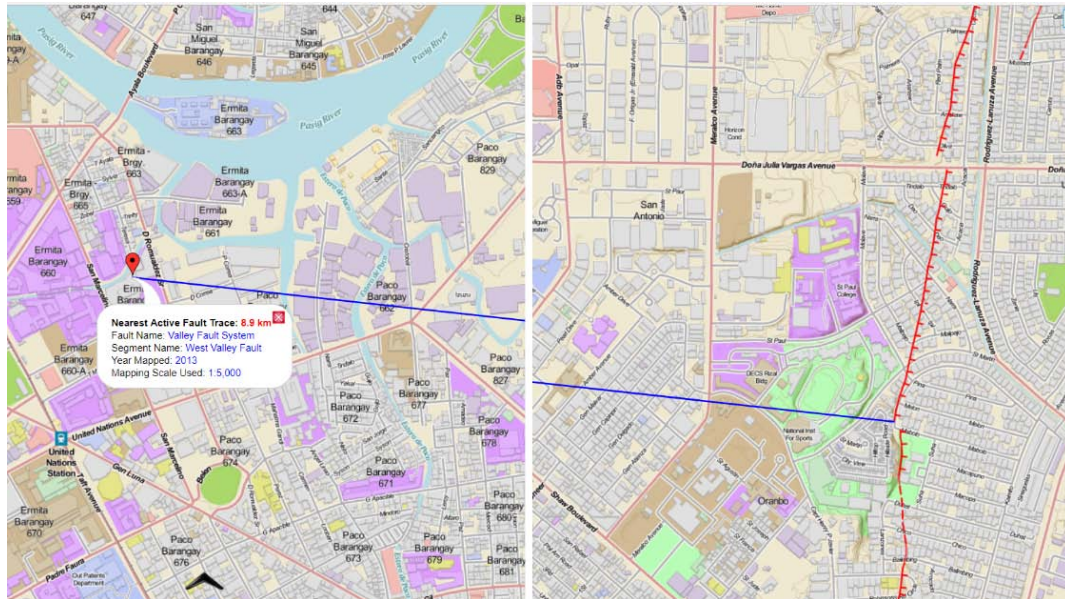
Fig. 1.1 : The nearest distance of the study from the immediate fault, i.e., the West Valley Fault, using PHIVOLCS Faultfinder App.

This study aimed to assess the seismic vulnerability of Adamson University buildings using its As-Built plan and Seismic fragility curve. Specifically, develop a set of as-built plan of the structure, generate sets of fragility curves by determining the damage indices and ranks of the structure from push over analysis and time history analysis using the following parameter: Displacement Ductility, Ultimate Ductility and Hysteretic Energy Ductility, provide with the vulnerability assessment of the structure by using the derived set of fragility curves, and evaluate the different derived seismic fragility curves of Adamson University buildings.

The scope of this study is to assess seismic vulnerability of Adamson University buildings. In Table 1.5, five (5) of the buildings of Adamson University was assessed.

Table 1 : Adamson University Buildings

	Name of Building	Year Erected	Main Structural Form
1.	CS Annex (Meralco Building)	1990	Reinforced Concrete Framing with Steel Roof trusses
2.	Cardinal Santos Building (formerly Main Meralco Bldg)	GF & 2F 1936 3F & 4F 1967	Reinforced Concrete Framing with Timber Roof trusses
3.	John Peyboyre Building	1990	Reinforced Concrete Framing with Steel trusses
4.	Francis Regis Clet Building	2000, 2002, 2012	Reinforced Concrete Framing with Roofdeck
5.	Saint Vincent Building	1932	Reinforced Concrete Framing with Timber Roof Trusses

Using Microsoft Excel and Matlab, the researchers obtained the parameters needed in deriving fragility curves. Furthermore, this study limited the seismic assessment methods used in creating fragility curves to Non-linear static analysis (Push Over Analysis) and Non-linear dynamic analysis (Time History Analysis) considering only shear as the mode of failure. Additionally, the structural performance of Adamson University buildings was evaluated under the effects of numerous normalized peak ground acceleration.

The ground motion data were acquired from Incorporated Research Institutions for Seismology (IRIS), the following data that used were listed below:

Table 2: Ground motion Data

Province of Epicenter Location	Date	Magnitude	PGA in g
Batangas	8 April 2017	6.0	0.20
Bohol	15 October 2013	7.2	0.22
Mindoro	15 November 1994	7.1	0.26
Moro Gulf	17 August 1976	8.1	0.50
North Luzon (Baguio)	16 July 1990	7.7	0.72
The Great Hanshin	17 January 1995	6.9	0.82
Ragay Gulf	17 March 1973	7.0	0.95
The Great Tohoku Kanto	11 March 2013	9.0	2.99

Other earthquakes after effects were not within the scope of the study. The basis for selecting these ground motion data was the relative location of earthquakes. This study was also be inclusive of probable cost of damage; reconstruction costs and retrofitting costs. This portion of the study will fall under Net Present Value Approach.

The structural model of Adamson University Buildings was based from As-Built Plans. These plans will be obtained by conducting As-Built/Field Surveys using surveying materials from Adamson University Engineering Department. The resulted As-Built Plans and the ground motion data, which will be acquired from the databases of PHIVOLCS and IRIS, will be used for the generation of seismic analysis.

In connection to modeling of structures, and performing the Nonlinear Static Analysis (Push-Over Analysis) and the Nonlinear Dynamic Analysis (Time History Analysis) the structural software to be used are SAP2000 to be able to determine the parameters needed such as; Ductility Factors, Damage Indices Factors, and Damage Rank, in attaining the fragility functions.

Thereupon, the data gathered from the analysis will then be used to finally plot the fragility curves. Using the fragility curves and by embodying the use of mathematical software, the seismic performance of the structure can be assessed.

II. Methodology

This study utilized the methods of Pushover Analysis (Non-linear static analysis) and Time History Analysis (Non-linear dynamic analysis). Requiso (2013) used the methods of Pushover Analysis (non-linear static analysis) in obtaining the yield displacement and the maximum displacement of the structure. Karim & Yamazaki (2001) used the methods of Time History Analysis (non-linear dynamic analysis) in obtaining the maximum displacement of the structure and for the derivation of the fragility curve. The software used for structural analysis, simulation and design was SAP2000.

The ground motion data used was acquired from the databases of PHIVOLCS (The Philippine Institute of Volcanology and Seismology) and IRIS (Incorporated Research Institutions for Seismology).

PHIVOLCS is a national institution that provides information about volcanic activities, tsunamis, and earthquakes. This institution was mandated to alleviate disasters that may arise from the said activities. While IRIS, is an educational center and a multi-institutional research having investigators from several consulting companies, universities, researchers at different state that provide contributions to; a performance-based earthquake engineering, seismology, risk management and etc., research programs.

This research was designed as shown in Figure 3.1 which intent to generate sets of fragility curves by determining the different parameters from performing the Nonlinear Static Analysis (Push-Over Analysis) and Nonlinear Dynamic Analysis (Time History Analysis) using structural software, SAP 2000. The created As-Built plan was used as the basis for the structural model of Adamson University Buildings.
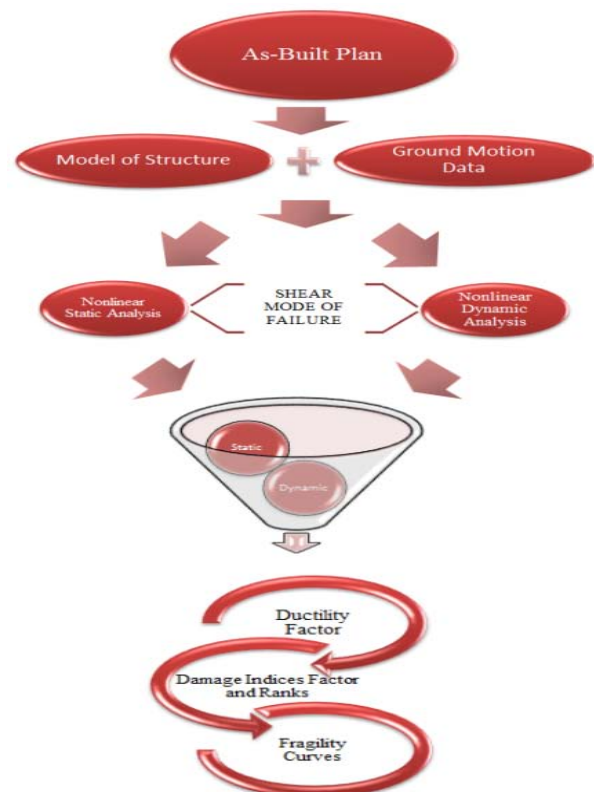

Fig. 3.1: Research Design

In every ground motion data's peak ground accelerations (PGA), the damage indices, one of the parameters, was obtained from 0.10g and 2.0g. In addition, the damage ranks were used as frequencies in solving the occurring probability for different peak ground acceleration (PGA) values.

The data gathered from the probability of occurrence and damage index were used to compute the mean and the standard deviation for the fragility analysis using the lognormal equation.

The seismic fragility curves were developed by plotting the cumulative lognormal probability versus the peak ground acceleration for every damage level.

In forming the As-Built plan for this study. The researchers used unique tools such as Total Station, a surveying measuring tool needed for the determination of the measurements needed for the As-Built Plan and Rebound Hammer to determine the compressive strength of the concrete used in the structure.

Consequently, the researcher used engineering software, Microsoft Excel, SAP2000, AutoCAD, MATLAB to generate the data and a 3D Model of the structure needed for the development of fragility curve.

A Schmidt hammer, also known as a Swiss hammer or a rebound hammer, is a device to measure the elastic properties or strength of concrete or rock, mainly surface hardness and penetration resistance. Ernst Schmidt, a Swiss engineer, invented it. Proceq and TQC worldwide distributed the Schmidt hammer. The hammer measures the rebound of a spring-loaded mass impacted against the surface of the sample. The test hammer will hit the concrete at a defined energy. Its rebound is dependent on the hardness of the concrete and measured by the test equipment. By reference to the conversion chart, the rebound value used to determine the compressive strength. When conducting the test, the hammer was held at right angles to the surface, which in turn should be flat and smooth. The rebound reading affects the orientation of the hammer, when used in a vertical position (on the underside of a suspended slab for example) gravity will increase the rebound distance of the mass and vice versa for a test conducted on a floor slab. The Schmidt hammer is an arbitrary scale ranging from 10 to 100. Schmidt hammers are available from their original manufacturers in several different energy ranges. These include: (i) Type L-0.735 Nm impact energy, (ii) Type N-2.207 Nm impact energy; and (iii) Type M-29.43 Nm impact energy.

The test is also sensitive to other factors:

- Local variation in the sample. To minimize this, it was recommended to take a selection of readings and take an average value.

- Water content of the sample, a saturated material gives different results from a dry one.

Prior to testing, the Schmidt hammer was calibrated using a calibration test anvil supplied by the manufacturer for that purpose. Twelve readings should be taken, dropping the highest and lowest, and then the average of the ten remaining has been taken. Using this method of testing classed as indirect as it does not give a direct measurement of the strength of the material. It simply gives an indication based on surface properties, it is only suitable for making comparisons between samples. This test method for testing concrete is governed by ASTM C805. ASTM D5873 describes the procedure for testing of rock.

In evaluation of the fragility curves, it was observed that there was an increase in every damage rank from various peak ground acceleration. It was also seen if there were a low, moderate or high possibility of structure to be damaged at 0.4g peak ground acceleration as the requirement of the National Structural Code of the Philippines. This also suggest if the structure can resist the different ground motion since a larger earthquake shaking is required to cause a significant damage.

This also generate a seismic report showing the possible damage that might occur within the structure. There is a recommending retrofitting scheme if the structure was subjected to 0.4g ground motion data considered it a high probability of exceedance of damage.

As shown in the Figure, the as built plan was first to establish with the used of as-built plans survey and non-destructive test to assess the seismic vulnerability of the building. These as-built plans were used in deriving fragility curves. Thereafter, the assessment of seismic vulnerability of the building has been performed.

Fig. 3.2.1.1: Research Design of As-Built Structural Modeling
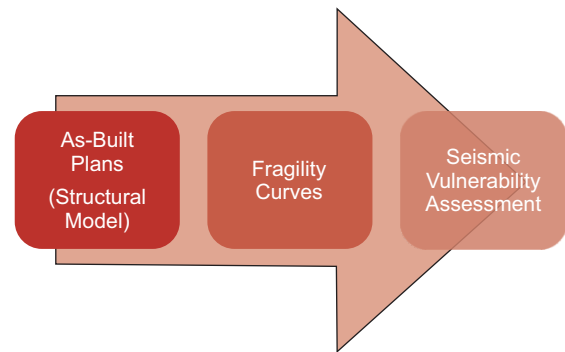
The structural model of Adamson University Buildings is subjected to both nonlinear dynamic analysis and nonlinear static analysis to assess the structure by attaining the fragility curves. In this study, the ratio of normalized peak ground acceleration and the original peak ground acceleration shall be multiplied to the ground motion records. The Normalized Ground

Motion data is scaled up or down from the original ground motion data. (Karim & Yamazaki, 2001).

$$\ddot{u}_{NEW} = A_0 \ddot{u}_{SOURCE} \qquad (3.1)$$

Where:

$\ddot{u}_{NEW} = A_0 \ddot{u}_{SOURCE}$ = the normalized ground motion data

$\ddot{u}_{SOURCE}$ = the source ground motion data

$A_0 = \dfrac{PGA_{normalized}}{PGA_{source}}$ = a coefficient factor to normalize the source of ground motion

The data gathered from two nonlinear analysis were used in determining the maximum displacement for the static and dynamic case. Furthermore, with the use of SAP2000, the yield displacement for static and dynamic case was determined. Moreover, the results was used for the computation of ductility factors by the following formulas (Karim & Yamazaki, 2001):

Displacement Ductility (μd):

$$\mu_d = \frac{\delta_{max}^{dynamic}}{\delta_y} \qquad (3.2)$$

where:

$\delta_{max(Dynamic)}$ = maximum displacement at the hysteresis model (dynamic)

δ_y = yield displacement from the push-over curve (static)

Ultimate Ductility (μ_u):

$$\mu_{u=\frac{\delta max \,(static\,)}{\delta y \,(static\,)}} \qquad (3.3)$$

where:

$\delta_{max(static)}$ = displacement at maximum reaction at the push over curve (static)

δ_y = yield displacement from the push-over curve (static)

Hysteretic Energy Ductility (μ_h):

$$\mu_h = \frac{E_h}{E_e} \qquad (3.4)$$

Where:

E_h = hysteretic energy (area under the hysteresis model)

E_e = yield energy (area under the push-over curve (static) but until yield point)

The damage indices were calculated and calibrated to their respective rank with the used of the computed values of ductility factors from the equations (3.2),(3.3), and (3.4). The computed values of damage indices was used to obtain the number of occurrence of each damage rank displayed in Figure 3.4.5.1. The formula for the computation of Damage Index was shown in equation (3.5).

$$DI = \frac{\mu_d + \beta \mu_h}{\mu_u} \qquad (3.5)$$

where:

β – Cyclic Loading Factor taken as 0.10 for buildings

Table 3.4.5.1: Damage Index and Damage Rank Relationship

(HAZUS, 2003)

Damage Index (DI)	Damage Rank (DR)	Definition
0.00 < DI ≤ 0.14	D	No damage
0.14 < DI ≤ 0.40	C	Slight damage
0.40 < DI ≤ 0.60	B	Moderate damage
0.60 < DI ≤ 1.00	A	Extensive damage
DI ≥ 1.00	As	Complete damage
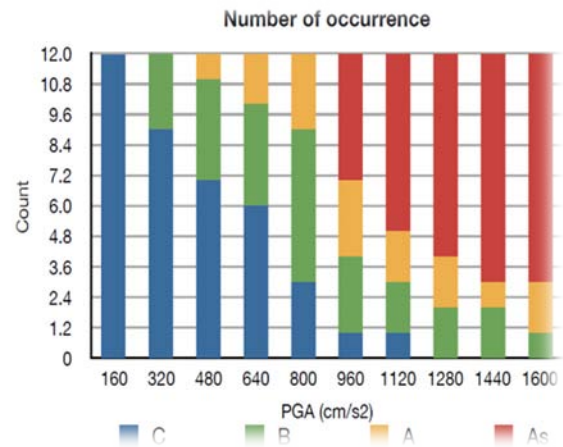
Fig. 3.4.5.1: Number of occurrence in each damage rank (Requiso, 2013)

The damage ratio is the number of occurrence of each damage rank (no, slight, moderate, extensive, and complete) divided by the total number of records. It was plotted against the natural logarithm *(ln)* of (PGA) to determine the mean and standard deviation that will be used to construct the fragility curves.

The parameters such as the mean and standard deviation were needed to construct the fragility curves. These parameters were obtained by plotting the value of damage ratio against the natural logarithm of PGA on a lognormal probability paper. Upon obtaining the values of the standard deviation and mean, equation (3.6) was used to compute for the cumulative probability of occurrence (Pr) of the damage equal or higher than the damage rank.

$$Pr = \Phi\left[\frac{\{\ln(X) - \lambda\}}{\xi}\right] \qquad (3.6)$$

Where:

Pr – Cumulative Probability

Ø – Standard Normal Distribution

X – Peak Ground Acceleration

λ – Mean

ξ – Standard Deviation

Maximum likelihood estimation is a method that determines values for the parameters of a model. The parameter values are found such that they maximize the likelihood that the process described by the model produced the data that were actually observed (Brooks-Bartlett, 2018).

The probability density of observing a single data point x, which is generated from a Gaussian distribution is given by:

$$P(x; \mu, \sigma) = \frac{1}{\sigma\sqrt{2\pi}} \exp\left(-\frac{(x-\mu)^2}{2\sigma^2}\right)$$

The semi colon used in the notation P(x; μ, σ) is there to emphasize that the symbols that appear after it are parameters of the probability distribution. So it shouldn't be confused with a conditional probability (which is typically represented with a vertical line e.g. P(A| B)).

This family of distributions has two parameters: $\theta = (\mu, \sigma)$; so we maximize the likelihood,

$$\mathcal{L}(\mu, \sigma) = f(x_1, \ldots, x_n \mid \mu, \sigma)$$

, over both parameters simultaneously, or if possible, individually.

Since the logarithm function itself is a continuous strictly increasing function over the range of the likelihood, the values which maximize the likelihood will also maximize its logarithm (the log-likelihood itself is not necessarily strictly increasing). The log-likelihood can be written as follows:

$$\log\left(\mathcal{L}(\mu, \sigma)\right) = -\frac{n}{2}\log(2\pi\sigma^2) - \frac{1}{2\sigma^2}\sum_{i=1}^{n}(x_i - \mu)^2$$

(Note: the log-likelihood is closely related to information entropy and Fisher information.)

We now compute the derivatives of this log-likelihood as follows.

$$0 = \frac{\partial}{\partial\mu}\log\left(\mathcal{L}(\mu, \sigma)\right) = 0 - \frac{-2n(\bar{x} - \mu)}{2\sigma^2}.$$

This is solved by

$$\hat{\mu} = \bar{x} = \sum_{i=1}^{n}\frac{x_i}{n}.$$

This is indeed the maximum of the function, since it is the only turning point in μ and the second derivative is strictly less than zero. Its expected value is equal to the parameter μ of the given distribution,

$$\mathrm{E}\left[\hat{\mu}\right] = \mu,$$

which means that the maximum likelihood estimator $\hat{\mu}$ is unbiased.

Similarly we differentiate the log-likelihood with respect to σ and equate to zero:

$$0 = \frac{\partial}{\partial\sigma}\log\left[\left(\frac{1}{2\pi\sigma^2}\right)^{n/2}\exp\left(-\frac{\sum_{i=1}^{n}(x_i - \bar{x})^2 + n(\bar{x} - \mu)^2}{2\sigma^2}\right)\right]$$

$$= \frac{\partial}{\partial\sigma}\left[\frac{n}{2}\log\left(\frac{1}{2\pi\sigma^2}\right) - \frac{\sum_{i=1}^{n}(x_i - \bar{x})^2 + n(\bar{x} - \mu)^2}{2\sigma^2}\right]$$

$$= -\frac{n}{\sigma} + \frac{\sum_{i=1}^{n}(x_i - \bar{x})^2 + n(\bar{x} - \mu)^2}{\sigma^3}$$

which is solved by

$$\hat{\sigma}^2 = \frac{1}{n}\sum_{i=1}^{n}(x_i - \mu)^2.$$

III. RESULTS AND DISCUSSION

It shows the structural plans, structural models, and rendered model using AutoCAD and SAP200 of Adamson University Buildings. In this floor plan, the distances of every column are shown, also the total length of the part of the building. These distances are easily measured by laser meter and every measurement, dimensions of the column are also taken. Thicknesses of the walls are also measured by the use of steel tape. These processes were repeated room by room and in every floor until all the data were completed. Using the data gathered, all the floor plans were drawn by the use of AutoCAD 2013, and the as-built plan was already developed.

CS Annex (Meralco) Building:
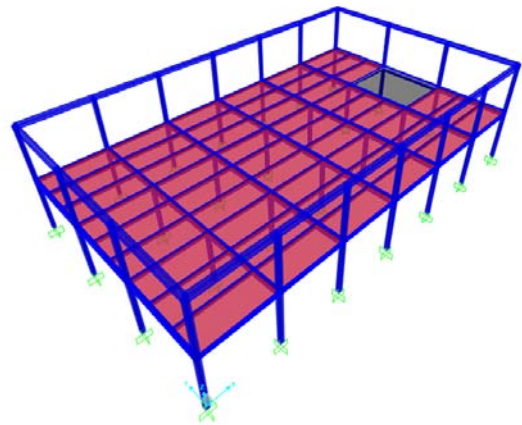
Cardinal Santos Building:
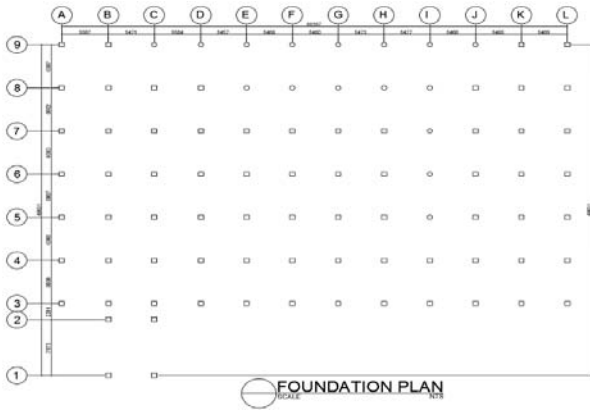
Fig. 4.1.5: Foundation Plan of CS Building

Fig. 4.1.6 2F: Plan of CS Building

Fig. 4.1.7 3F: Plan of CS Building

Fig. 4.1.8 4F: Plan of CS Building

Fig. 4.1.9: Truss Elevation View

John Peyboyre Building:
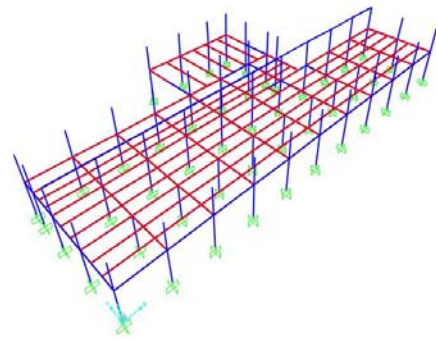
Fig. 4.1.10: Three – Dimensional Model of John Perboyre Building

Father Regis Clet Building:
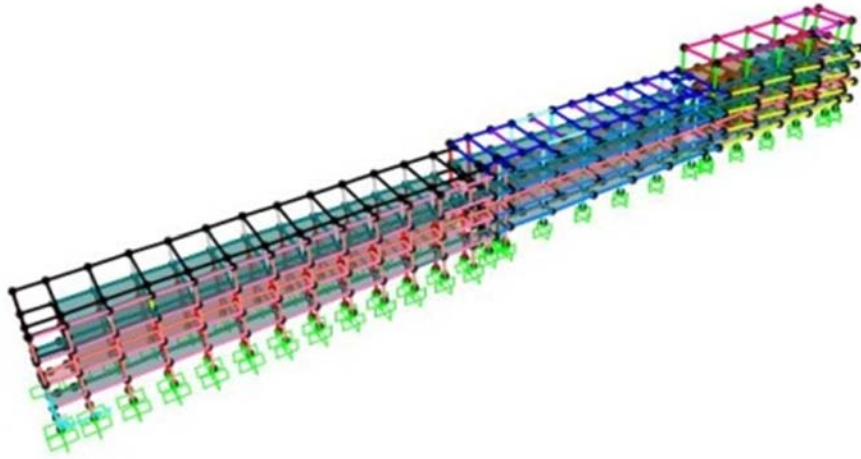
Fig. 4.1.26: Structural Model of FRC Building

Saint Vincent Building:

CS Annex (Meralco Building):

Fig. 4.2.1: Pushover Curve X-Direction

Fig. 4.1.27: Three – Dimensional Model of Saint Vincent Building

IV. PUSHOVER CURVE

This pushover curve represents the relationship between the displacement and the base force. The following figures show that the displacement and the base force are directly proportional. The straight line represents the limit of the structure to resist the maximum base force applied. The last point before the linear graph changes into a curved graph is the yielding point. And beyond that curved graph, the structure was completely damaged. The curved part in these graphs are not noticeable because the values and differences are small.

Fig. 4.2.2: Pushover Curve Y-Direction

Cardinal Santos Building:
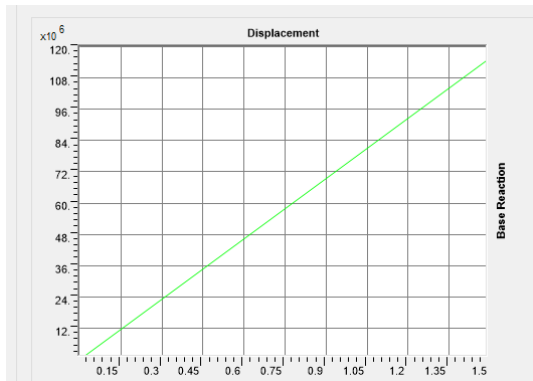
Fig. 4.2.3.: Pushover Curve (X-direction)
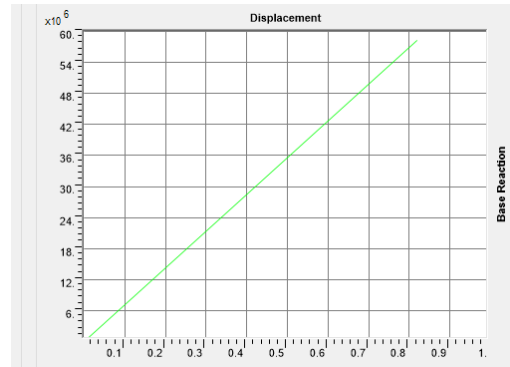
Fig. 4.2.4.: Pushover Curve (Y-direction)

John Peyboyre Building:
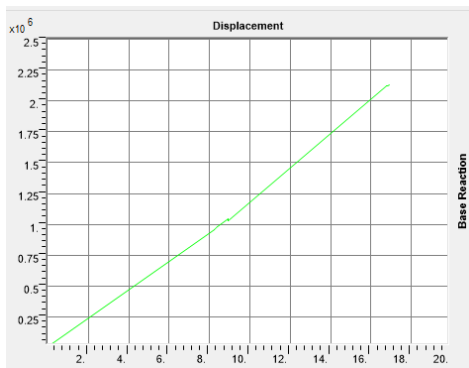
Fig. 4.2.5.: Pushover Curve X-direction

Fig. 4.2.6.: Pushover Curve Y-direction

FRC Building:
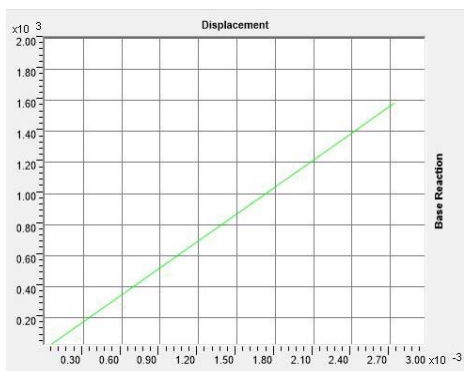
Fig. 4.2.7.: Pushover Curve in X - Direction
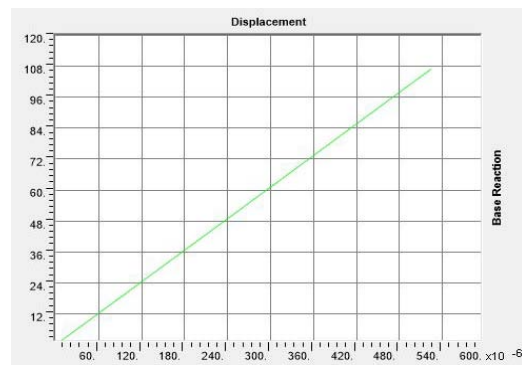
Fig. 4.2.8.: Pushover Curve in Y – Direction

Saint Vincent Building:
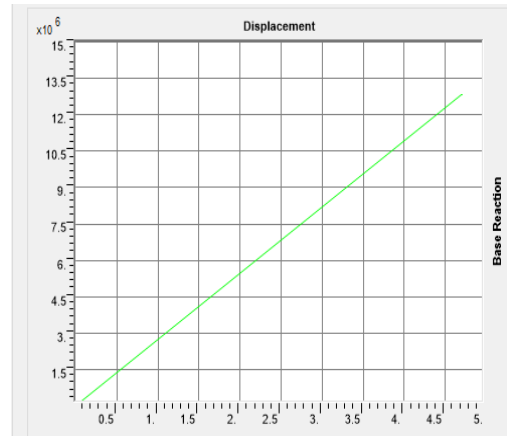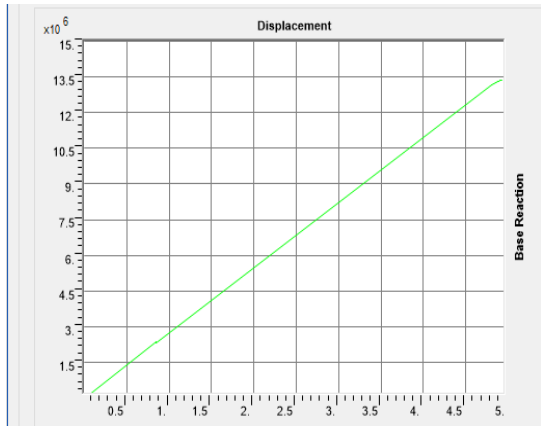
Fig. 4.2.9.: Sample Pushover Curve (X –axis)

Fig. 4.2.9.: Sample Pushover Curve (Y –axis)

The following charts are the plots that demonstrates how the fitted fragility curves using MLE to find the statistical parameters for the lognormal distribution function. The red dots are the plot of the cumulative values of probability of occurrence based on the damage ratios.

CS Annex (Meralco) Building:
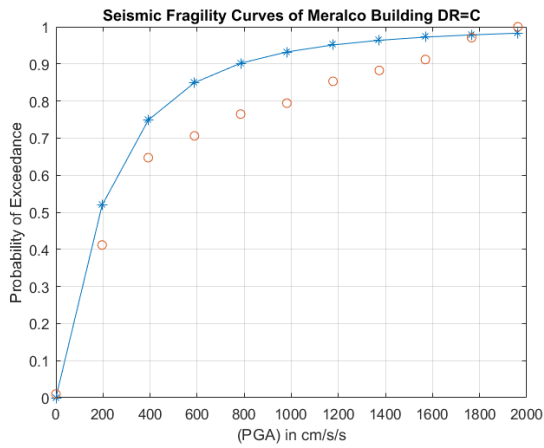
(a)
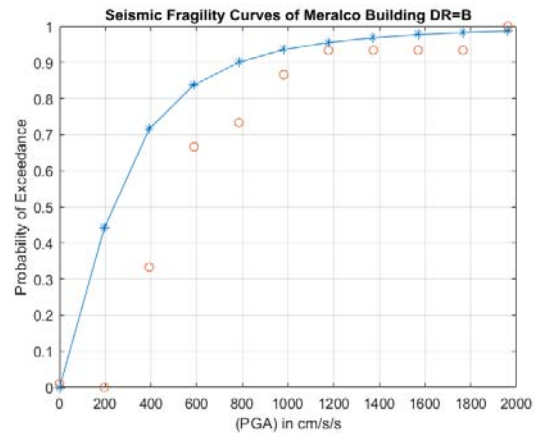
(b)
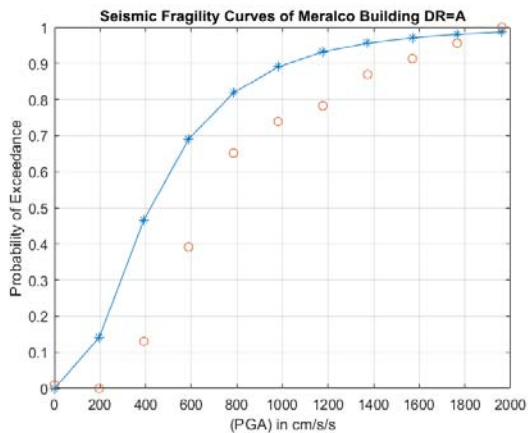
(c)
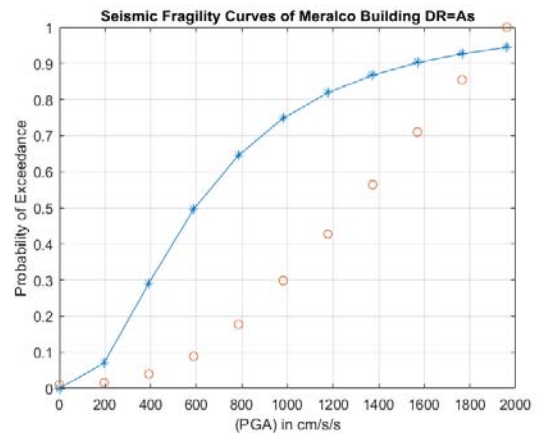
(d)

Fig. 4.5.1.1

Cardinal Santos Building:
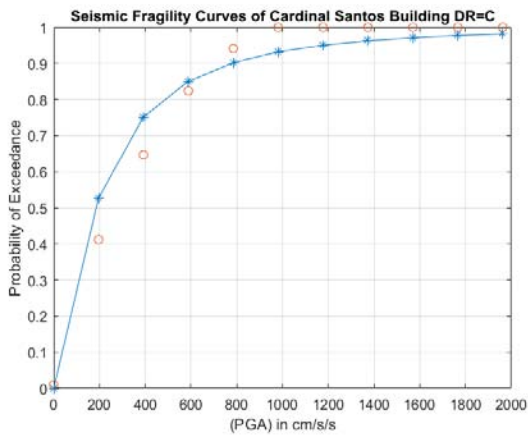
(a)
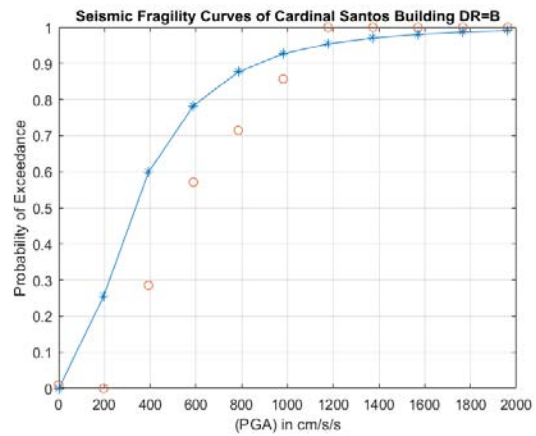
(b)
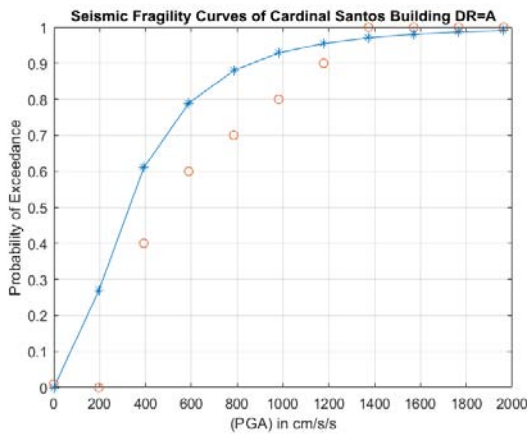
(c)
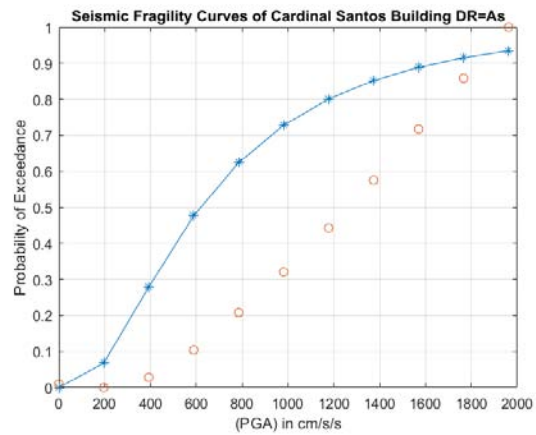
(d)

Fig. 4.5.1.2

John Peyboyre Building:
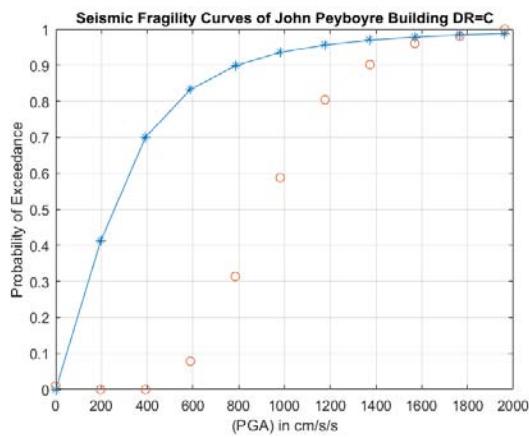
(a)
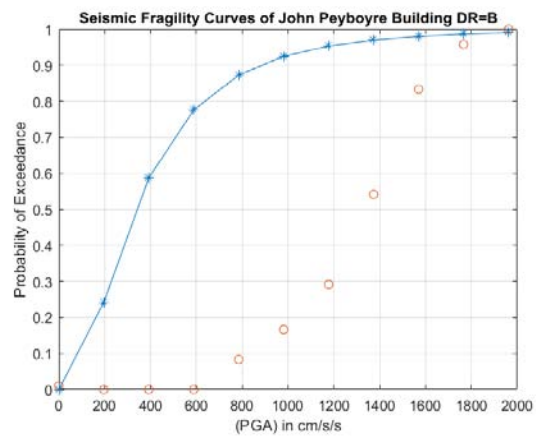
(b)
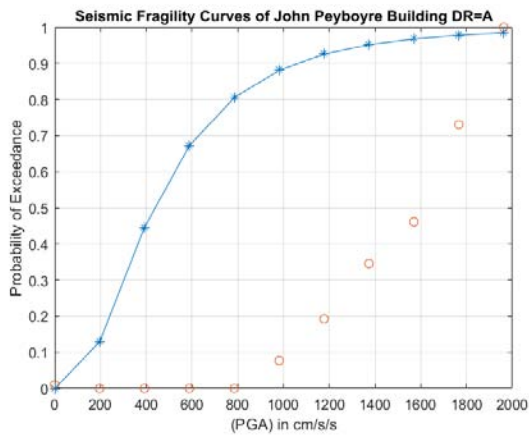
(c)
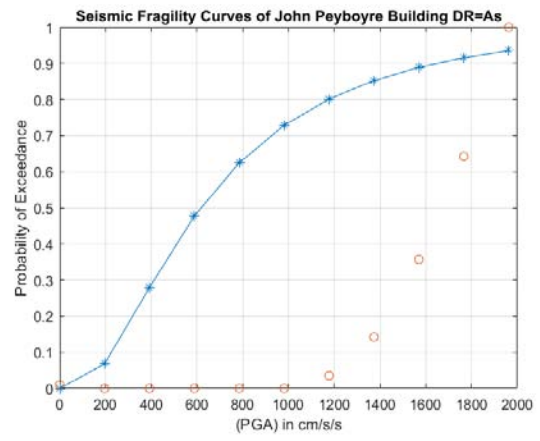
(d)

Fig. 4.5.1.3

FRC Building:

(a)

(b)

(c)

(d)

Fig. 4.5.1.4

40

Saint Vincent Building:

(a)

(b)

(c)

(d)
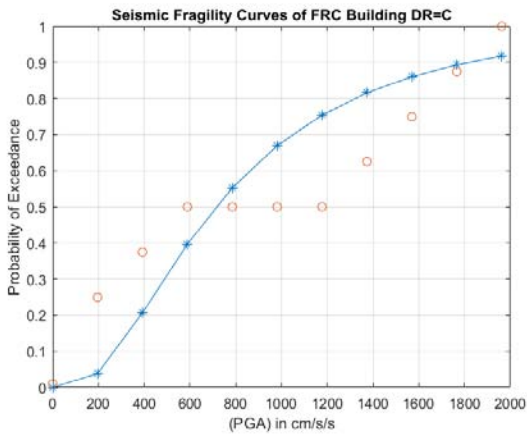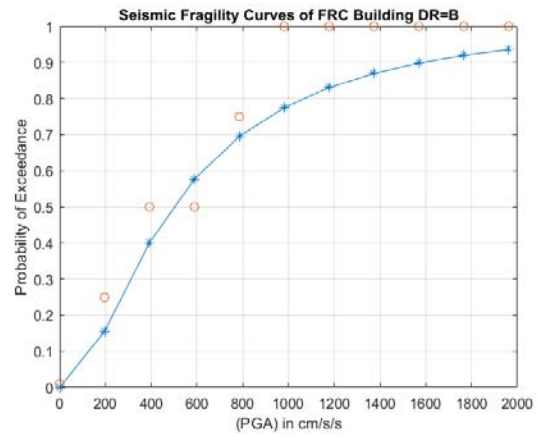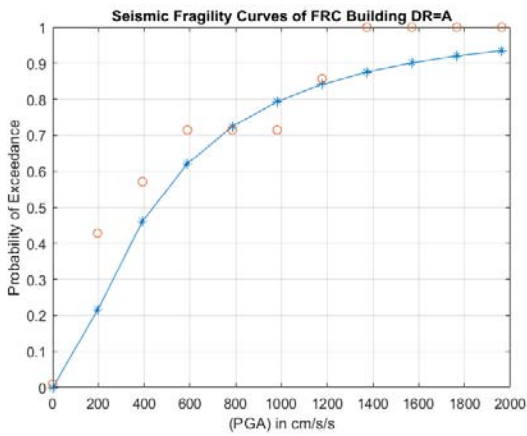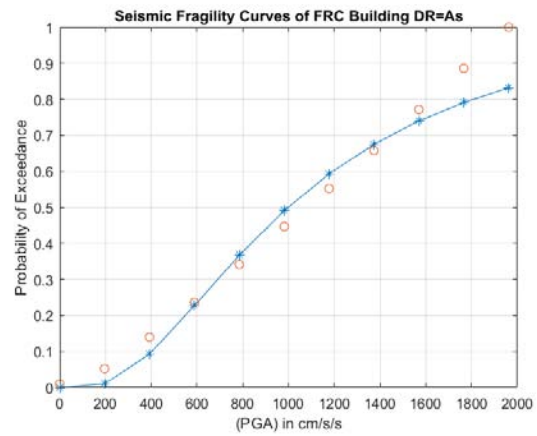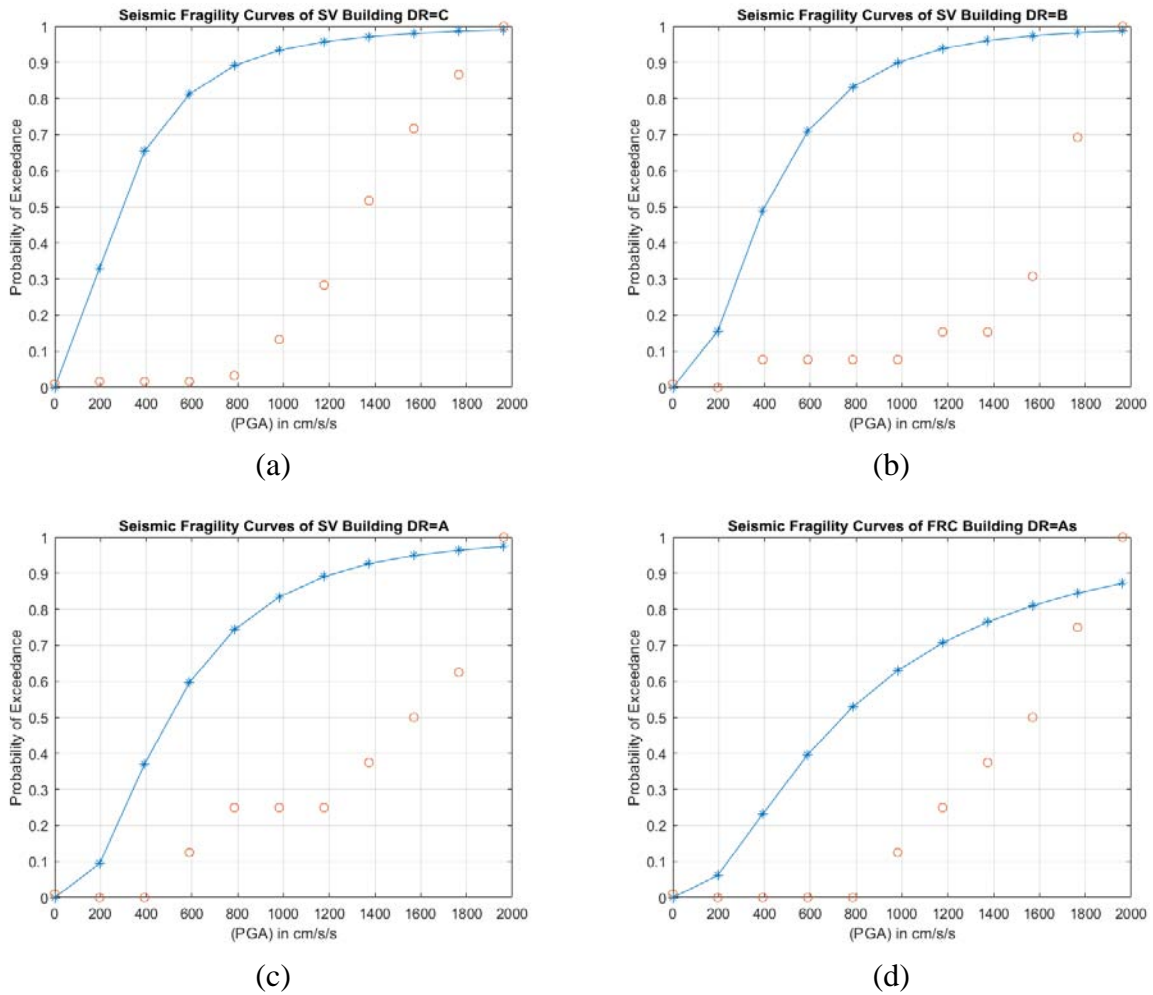
Fig. 4.5.1.5

The following charts are the plot of seismic fragility curves for each building. The different damage ranks can compared. In practice, the damage rank "D" or "No Damage" must not be included in the fragility analysis.
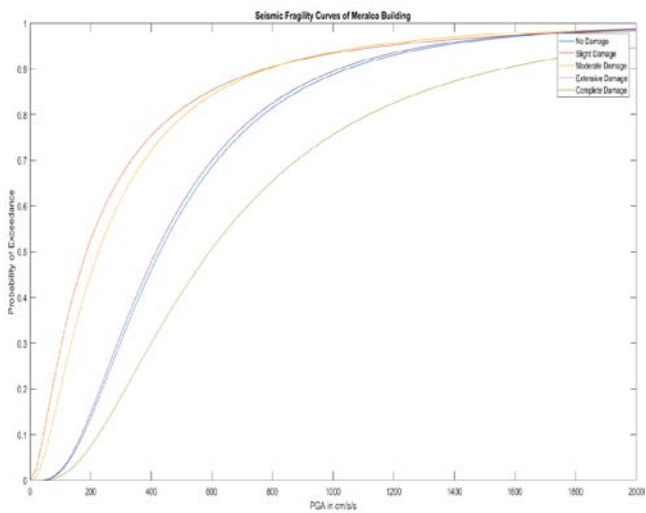
Fig. 4.5.2.1
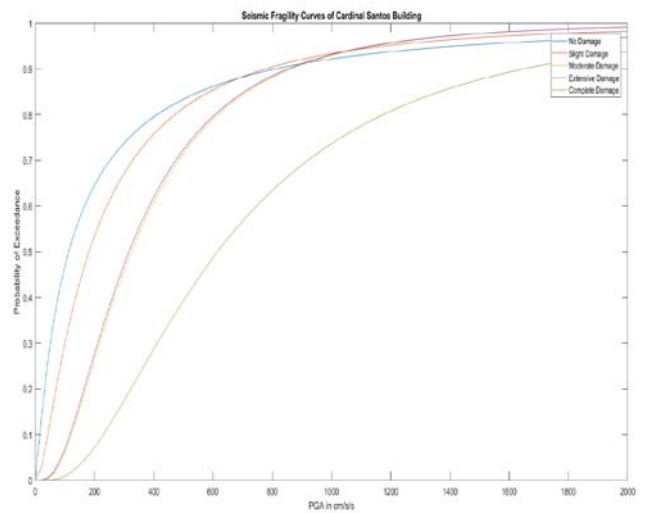
Fig. 4.5.2.2
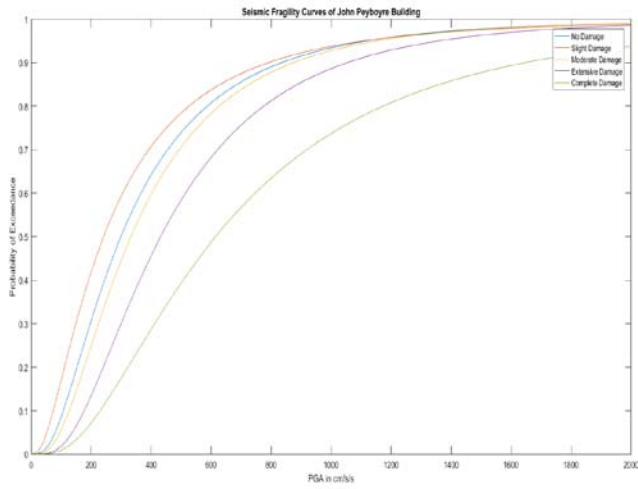
Fig. 4.5.2.3
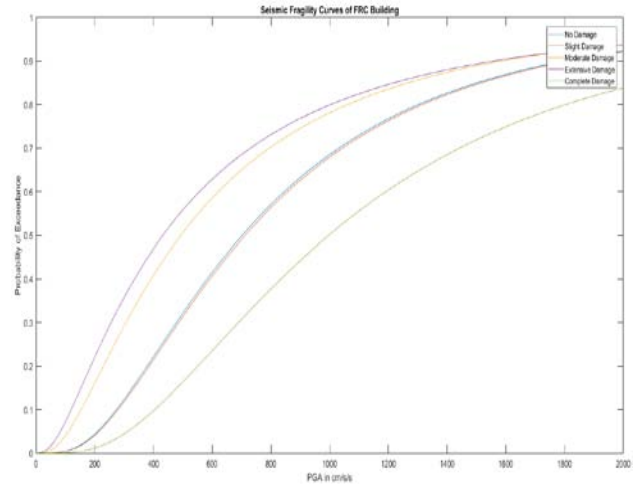
Fig. 4.5.2.4
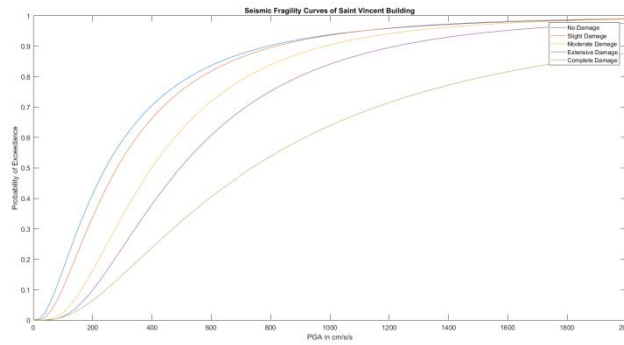
Fig. 4.5.2.5

In order to compare each building's seismic performance, the following charts are needed to which structure is evaluated as least and most resilient as per damage rank.
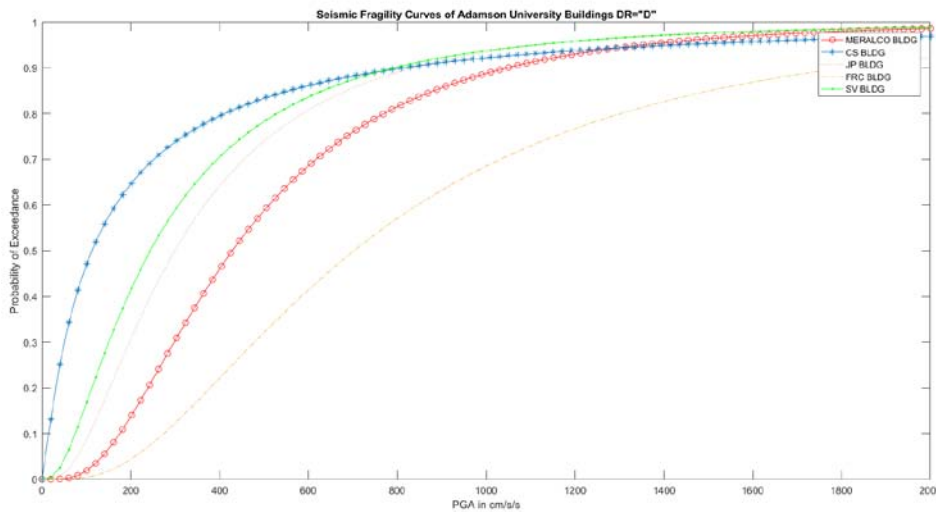
Fig. 4.5.3.1
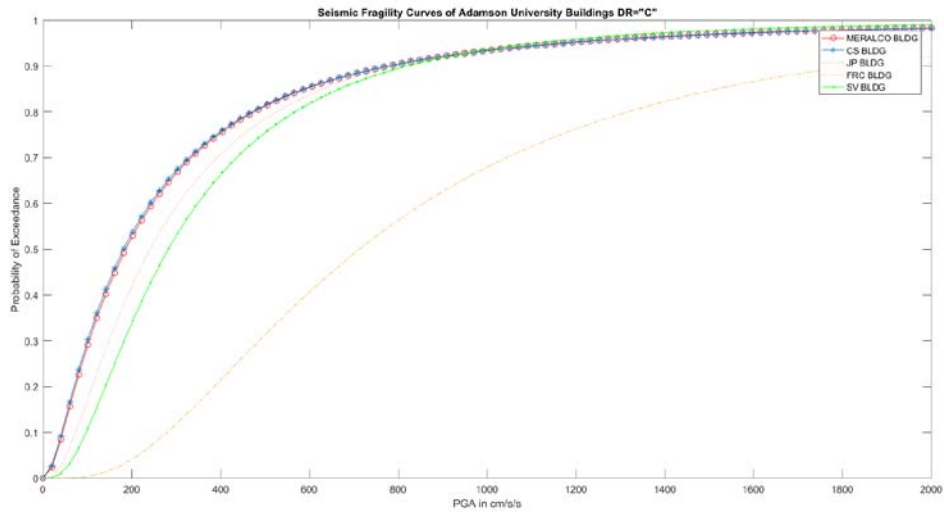
Fig. 4.5.3.2
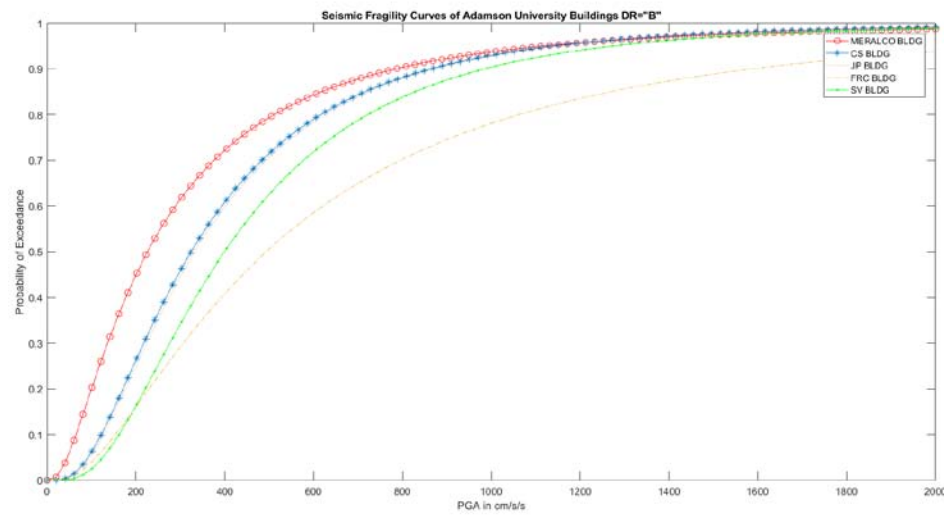
Fig. 4.5.3.3
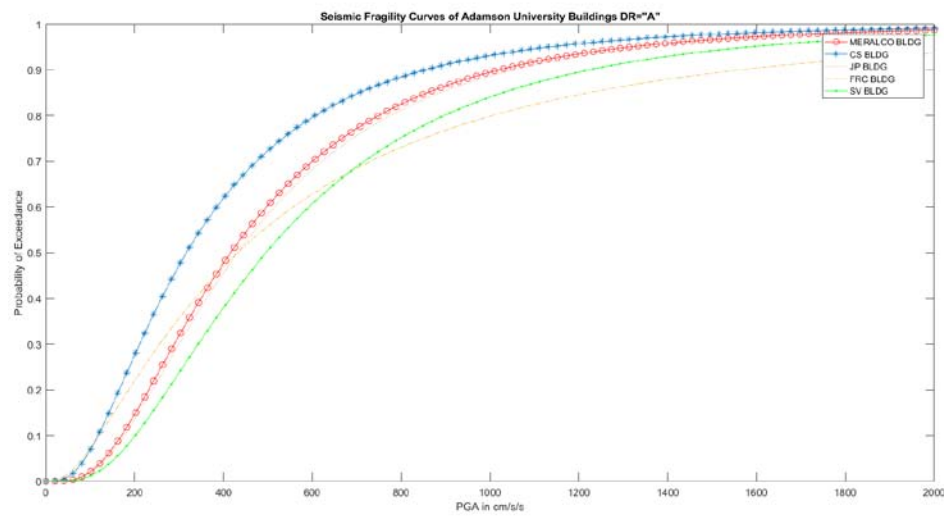
Fig. 4.5.3.4
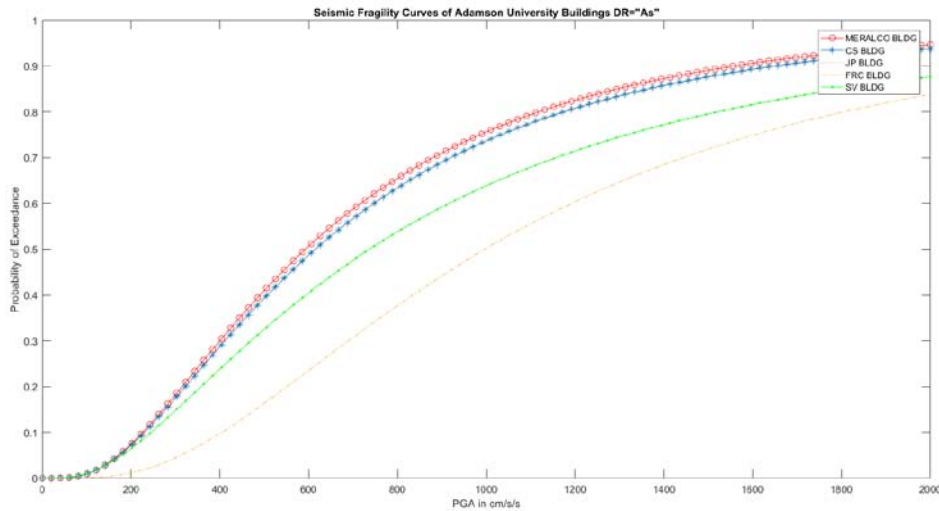
Fig. 4.5.3.5

The table summarizes the values of probability of exceedance at PGA=0.4g or 392.4 cm/s/s for each building per damage rank of "C" or "Slight Damage" to "As" or "Complete Damage".

Table 4.7.1: Summary of Probability of Exceedance of Adamson University Buildings for every Damage Rank

Building	C	B	A	As
CS Annex (Meralco) Building	74.85	71.53	46.66	29.17
CS Building	75.19	59.85	60.99	27.89
JP Building	70.03	58.72	44.58	27.93
FRC Building	20.68	40.07	46.15	9.29
SV Building	65.42	49.01	37.01	23.18

In section 2.3.2 of the National Structural Code of the Philippines (NSCP) Volume 1, the following figure is excerpted:
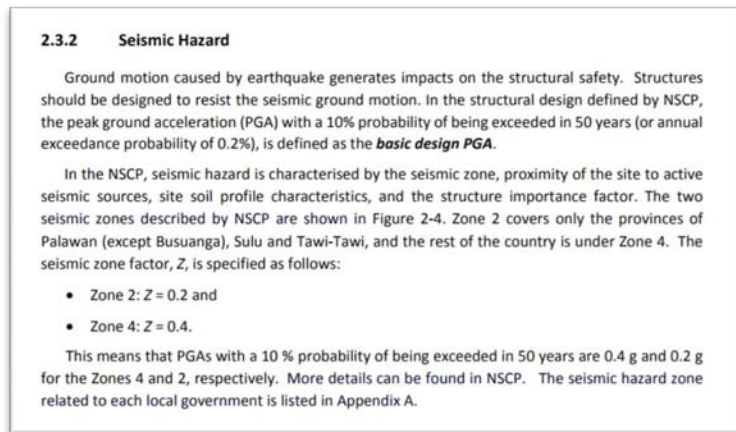
Fig. 4.7.1.: NSCP Specifications on 0.4g PGA for Zone 4

In addition, the Structural Engineers Association of California (SEAOC) has the following excerpt:

"A structure with a 30 or more years of lifespan is NOT SAFE when subjected to a seismic event of 10% probability of exceedance of collapse or total damage. The structure being more than 50 years old is vulnerable to large magnitude earthquakes."

Based on the table and the SEAOC specifications, only the Father Regis Clet (FRC) Building can still withstand a 0.4g PGA and meets the code specs, that is, a probability of exceedance (POE) of 9.29%. The rest of the buildings are vulnerable to a 0.4g PGA, that is more than twice that of SEAOC specifications.

To summarize, these results indicate that the most probable damage that the structure will sustain will range from Extensive damage to Complete Damage which is significantly below the standards imposed as per NSCP 2010 which states that the structural integrity of the building must withstand 40% of the gravitational acceleration as the peak ground acceleration. Based from the resulting fragility curves, the structure is unsafe for occupancy when subjected to seismic activity, considering only base shear as the mode of failure.

V. Conclusion

The development of as-built plan is only limited in the concrete works of the building. The developed as-built plan includes of elevations, floor plans and other structural plans.

The accuracy of the As-built plans made is mainly dependent on two factors, namely the precision and accuracy of the equipment and human errors. Usage of laser measuring devices and available digital counterparts of surveying equipment yields more reliable and consistent values thus reducing any deviation from the actual measurements.

Based from the ductility parameters derived from undergoing nonlinear static and nonlinear dynamic analyses, all the Damage ranks that were observed in the fragility curve along the x-axis at its longitudinal section are No damage, Slight Damage, Moderate Damage, Extensive Damage and Complete Damage. The probability of sustaining Slight Damage is small until the 0.1g PGA which tends to increase well beyond it. The opposite was observed on the y-axis or transverse section of the building due to having a much steeper incline on the Slight Damage curve suggesting a high probability of transitioning to a higher damage state, making the structure more susceptible to damage when subjected to strong seismic activity along the transverse section.

From what can be gathered from the generated fragility curve, the building has a probability of exceedance for corresponding damage rank. In line with these, it can be inferred that the Adamson University Buildings did not meet the requirement that it must withstand a peak ground acceleration of about 40% of the gravitational acceleration as per National Structural Code of the Philippines 2010.

VI. Recommendation

Using the set of data and conclusions derived from this study, the authors recommended the following:

This study only covers the susceptibility of members to seismic damage and does not deal with the methods to be used for retrofitting (if possible) along with the corresponding cost due to loss in serviceability of the structure and the cost of retrofitting depending on the level of damage. For this reason, incorporation of the Fragility curve reassessed from an economic point of view would develop a more conclusive set of guidelines as to how to approach this problem.

The researchers would recommend doing tests to determine whether the FRC building is applicable for any retrofitting measures that would allow it to withstand the level of ground motion as per NSCP 2010.

This type of study is only one of the other possible methods of seismic vulnerability assessment. Usage of other methods such as the Monte Carlo Simulations and Latin Hypercube Sampling would be recommended to determine the discrepancy in the results.

The Administrator has to expose and updates through seminars/trainings the staffs of Physical and Facility Office (PFO) on the latest technology in Structural Reliability, specifically, structural assessments of existing structures, by attending the nation's recognized organization, such as Philippine Institute of Civil Engineers and Association of Structural Engineers of the Philippines, Inc.

VII. Implications

Modeling of Structural Fragility relies on damage data generated either from empirical or mechanical methods (Cobum& Spence, 2002). Damage probability functions can be derived from documented post-earthquake damage observations, as with the case in empirical methods (Rota et al,2008). Damage estimates can also be derived using mechanical models such as the use of nonlinear analysis in the area of less earthquake prone regions due to lack of observational data (Karbassi and Nollet, 2013). As for this study, the latter was done using PGA as the primary parameter.

Unlike the Richter Scale or the Moment Magnitude Scale, PGA is not a measure of the total energy of an earthquake but as a measure of the intensity of the acceleration at a geographic point. Hence, PGA is not indicative of any level of magnitude but the probability of achieving higher PGA repeatedly increases as the Magnitude increases. It can be inferred that the probability of achieving a certain PGA repeatedly is directly proportional to the magnitude. Even in smaller magnitudes, high PGA's can be attained though repeated occurrence is much smaller (USGS, 1994).

PEAK GROUND ACCELERATION VS. MAGNITUDE
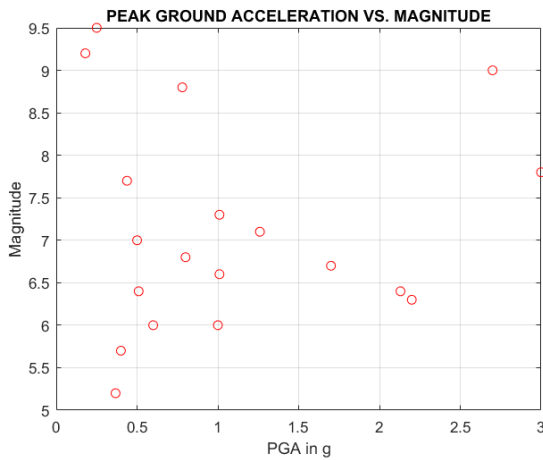
All form of matter obeys the most basic fundamental principle of physics which is Inertia. Anybody will continue to perpetuate its current state unless influenced by outside force. If it is moving at a certain direction at a certain speed, it will continue to do so unless acted upon by any force and will remain still if let be. Buildings, as it is made of matter, behave in a similar manner.

Earthquakes do not move in one direction, relative to a particular axis. Let us say that the X-axis is the longitudinal section of the building. Ground motion is not inclined to move only at one direction of the longitudinal section may it be to the left or to the right. It goes both ways. One can illustrate it when moving a piece of string back and forth. By doing so, wave like patterns are created. The dips in the waves indicate the sudden change in the direction of the acceleration while the amplitude of the wave is an indication of the magnitude of the change.

This is because of inertial lag. Inertial lag, by definition, is the delay in the response of a flow to the forces acting upon it. Mechanical response of any object is not instantaneous. No object is perfectly rigid. With regards to a structure, the bottom of the column is the first to respond to any form of ground movement before it reaches the top. But unlike the string, columns are sufficiently rigid and are essentially brittle. The greater the acceleration, the greater the inertial lag for every abrupt change in the direction of acceleration. The greater the inertial lag, the higher the probability of any of the structural member to break off.

As so, even though PGA isn't indicative of the total energy being introduced by the ground to the building, it can illustrate the level of damage the structure incurs as it achieves that level of acceleration. These damages accumulate throughout every abrupt periodic change in the direction of the ground acceleration until the accumulated damage exceeds its allowable limit subsequently causing its collapse.

Intensity on the other hand is essentially a qualitative scale of the level of damage which can be described in terms of perceived shaking or potential damage to manmade structures. But it does not indicate the amount of energy released during an earthquake unlike its Magnitude. Despite that, the amount damage structures sustain increases with the increasing magnitude, under the assumption of constant duration (USGS, 1994).

Below is an example of a proposed correlation of the Modified Mercalli Intensity Scale with PGA in Costa Rica.

Table 4.8.1: Proposed ranges of PGA for each Instrumental MMI in Costa Rica (Linkimer, 2008)

Intensity (MM)	PGA Max Range (cm/s^2)	PGA Max Range (% g)	PGA Max Range (cm/s^2)	PGA Max Range (% g)
II	< 4.9	< 0.5	< 5.6	< 0.6
III	4.9-13.3	0.5-1.4	5.6-15.0	0.6-1.5
IV	4.9-13.3	1.4-3.7	15.0-40.3	1.5-4.1
V	36.0-80.3	3.7-8.2	40.3-84.7	4.1-8.6
VI	80.3-146.7	8.2-15.0	84.7-139.6	8.6-14.2
VII	146.7-268.0	15.0-27.3	139.6-230.2	14.2-23.5

This correlation may serve as a good fit for structures in Costa Rica but this relationship is strictly limited to parameters obtained in that particular area. Even then, "locations within the same intensity area will not necessarily experience the same level of damage since damage depends heavily on the type of structure, the nature of the construction, and the details of the ground motion at that site" (USGS, 2012). Also, instrument-only based approach for Generalized

Damage Intensity relative to ground motion parameters such as PGA and PGV does not account for the structural characteristics of buildings and, therefore, may not provide useful information about the damage state of the built environment following an earthquake (Tan &Irfanoglu, 2012). Duration also contributes to the damage sustained in a structure. An earthquake could be relatively weak but occur at a long period of time effectively inducing the same amount of damage a

relatively strong earthquake can cause in a short amount of time.

Because of these variations, using Damage as an indicator of determining the threshold of Magnitude at which it will be considered unsafe will only yield a probabilistic range of Magnitudes. And in doing so, requires further study about all other parameters influencing the structural integrity of buildings in a particular area which is outside the coverage of this study. The scope of this study is only limited to the development of Fragility Curves to assess the seismic performance of the building in question using PGA as a parameter. That is to say, the purpose of this study is to create a probabilistic model of the level of observational/qualitative material damage in terms of its quantitative counterparts derived from Damage Indices using PGA and does not take in to consideration other factors that could influence the change in the actual incurred damage relative to the theoretical damage.

As it is usually the case in seismic damages, failure of a member is the product of all modes of failures. However, the mode of failure only considered in this study is limited to shear.

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Pushover Analysis of an OMRF Building Located in Dhaka

By Soumya Suhreed Das, Sudipta Roy & Adhora Tahsin

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Abstract- Tall and highrise buildings located in seismically vulnerable zones usually need to go through seismic evaluation to check its resilience against cyclic loading produced due to surface waves created by earthquakes. Large seismic waves create undulations in soils which drastically reduces the strength of foundations and ordinary moment resisting frames and the following aftershocks accelerate crack propagation of structural systems and dynamic overloading, leading to a heavy toll on lives. To protect buildings in dynamically active zones, moment resisting frames need seismic detailing alongside seismic testing. These paper deals with nonlinear dynamic analysis(pushover techniques) on a highrise building located in Dhaka city which was originally designed as a simple moment resisting frame, and necessary optimization of structural elements to improve its function against dynamic loading using the help from the BNBC code and the ETABS software.

Keywords: BNBC, pushover, OMRF, non-linear analysis, seismicity, plastic hinge, structural vibrations, capacity curve.

GJRE-E Classification: FOR Code: 290801

PUSHOVERANALYSISOFANOMRFBUILDINGLOCATEDINDHAKA

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Pushover Analysis of an OMRF Building Located in Dhaka

Soumya Suhreed Das α, Sudipta Roy σ & AdhoraTahsin ρ

Abstract- Tall and highrise buildings located in seismically vulnerable zones usually need to go through seismic evaluation to check its resilience against cyclic loading produced due to surface waves created by earthquakes. Large seismic waves create undulations in soils which drastically reduces the strength of foundations and ordinary moment resisting frames and the following aftershocks accelerate crack propagation of structural systems and dynamic overloading, leading to a heavy toll on lives. To protect buildings in dynamically active zones, moment resisting frames need seismic detailing alongside seismic testing. These paper deals with nonlinear dynamic analysis(pushover techniques) on a highrise building located in Dhaka city which was originally designed as a simple moment resisting frame, and necessary optimization of structural elements to improve its function against dynamic loading using the help from the BNBC code and the ETABS software.

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I. Introduction

Pushover is a static-nonlinear analysis method where a structure is subjected to gravity loading and a monotonic displacement-controlled lateral load pattern which continuously increases through elastic and inelastic behaviour until an ultimate condition is reached. Federal Emergency Management Agency (FEMA) and Applied Technical Council (ATC) are two agencies which formulated and studied nonlinear static/pushover analysis under seismic rehabilitation and protection guidelines, which followed documents FEMA 356 and ATC-40. Lots of researches have been made on this topic, and still numerous software are being developed every day for dynamic modelling of more complex structures. Dynamic analysis helps assess a structures' vulnerability against different site soil characteristics, and categorizes a moment resisting frame as ordinary, intermediate and special. Special moment resisting frame needs ductile reinforcing to be able to absorb more seismic shocks. Pushover techniques are almost similar to time history analysis which provides the structural dynamic response with time and it is different from the response spectrum analysis which is linear dynamic statistical analysis method measuring the contribution from each natural mode of vibration to indicate the likely maximum seismic response of an essentially elastic structure. Response-spectrum analysis provides insight into dynamic behaviour by measuring pseudo-spectral acceleration, velocity, or displacement as a function of structural period for a given time history and level of damping. It is practical to envelop response spectra such that a smooth curve represents the peak response for each realization of structural period. But unlike these two methods, nonlinear dynamic pushover is way better in analysing the actual behaviour of structures.

There are mainly two methods of this analysis-Displacement Coefficient and Capacity spectrum. BNBC equivalent static force is limited for structures having heights less than 20metres, which is not so rigorous in case of Pushover analysis. K. chopra and K. Goel [2] commented that MPA procedure with rigorous nonlinear response history analysis (RHA) demonstrates that the approximate procedure provides good estimates of floor displacements and story drifts, and identifies locations of most plastic hinges. However, regarding story drift, they concluded that all pushover analysis procedures considered do not seem to compute to acceptable accuracy local response quantities, such as hinge plastic rotations. Thus the present trend of comparing computed hinge plastic rotations against rotation limits established in FEMA-273 to judge structural performance does not seem prudent. R. Shahrin and T. Hossain[3] used masonry infilled walls for seismic performance evaluation against bare frame walls and found out that the former performed better in Pushover.

II. Analysis Works

To perform pushover a highrise building located at Niketan, Dhaka is chosen as a test subject. The test site soil was in S2 condition (a soil profile with dense and stiff soil condition where soil depth exceeds 61 metres). Normally, according to BNBC 2006 and ASCE code requirements, these soils are seismically efficient to absorb and control structural vibrations. Buildings built on these systems are seismically sufficient for a certain degree of shaking, if recurring earthquakes possess a magnitude more than richter scale 6.0 then seismic detailing and pushover analysis are required.

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According to BNBC 2014 article 2.5.14, For regular structures with independent orthogonal seismic-force-resisting systems, independent two-dimensional models may be used to represent each system. For structures having plan irregularities or structures without independent orthogonal systems, a three-dimensional model incorporating a minimum of three degrees of freedom for each level of the structure, consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis, shall be used. Where the diaphragms are not rigid compared to the vertical elements of the seismic-force-resisting system, the model should include representation of the diaphragm flexibility.

The lateral forces shall be applied at the mass center of each level(control point) and shall be proportional to the distribution obtained from a modal analysis for fundamental mode of response in the considered direction, and the lateral loads shall be increased incrementally in a monotonic manner. The analysis will be continued until the displacement of the control point is at least 150% of the target displacement. A bilinear curve shall be fitted to the capacity curve, such that the first segment of the bilinear curve coincides with the capacity curve at 60% of the effective yield strength, the second segment coincides with the capacity curve at the target displacement, and the area under the bilinear curve equals the area under the capacity curve, between the origin and the target displacement. The effective fundamental period and target displacement shall be expressed as-

$$T_e = T_1 \sqrt{\frac{V_1/\delta_1}{V_y/\delta_y}}$$

$$\delta_T = C_0 C_1 S_a \left(\frac{T_e}{2\pi}\right)^2 g$$

Where V1, δ1, T1 are determined for the first increment of lateral load. And spectral aceleration as well as coffiecient shall be calculated accordingly.

According to FEMA 356[4] seismic performance levels, structural response in divided into several categories: Immediate occupancy(IO), Life Safety(LS), Collapse Prevention(CP). When structure is at IO level, this level is without any damage(although some cracks might be seen near slab-column connection or drop panel location, minor cracking in columns-not visible). When the structure is at LS level, slabs sustain extensive cracking at connections (at drop panels), and flexure cracking is seen at the top of column which may necessitate retrofitting. And the final stage, CP causes extensive damage in diaphragms, and top of columns. So, for a structure to be seismically resilient, it needs to be in seismic performance level IO.
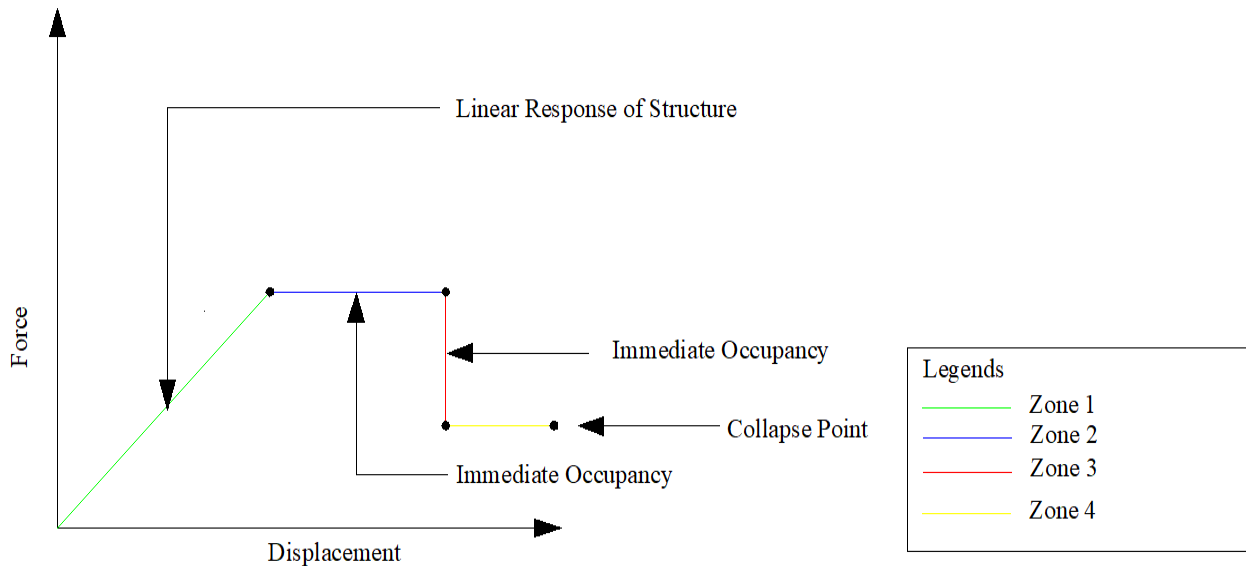
Fig. 1: Structural response curve due to dynamic loading

From force displacement curve of structural frames the following data can be found. With application of load, the dynamic response is linear upto certain point, then the structure enters the IO zone, and after that it enters the strain hardening zone and afterwards collapse. In ETABS 2015 or other versions, pushover analysis depicts these conditions in green, cyan, red and orange.

III. Plan Selection

A highrise residential apartment complex has been chosen as a model for Pushover analysis. This building is a G+10 storied building located in Mirpur, Dhaka-Bangladesh. Site soil condition is S2(strong soil upto necessary depth, also satisfactory for piling operation). Structural plan is regular with fourteen

number of columns. Necessary visual information regarding terrain condition, soil profile and building structural plan have been collected from computer aided drawing.
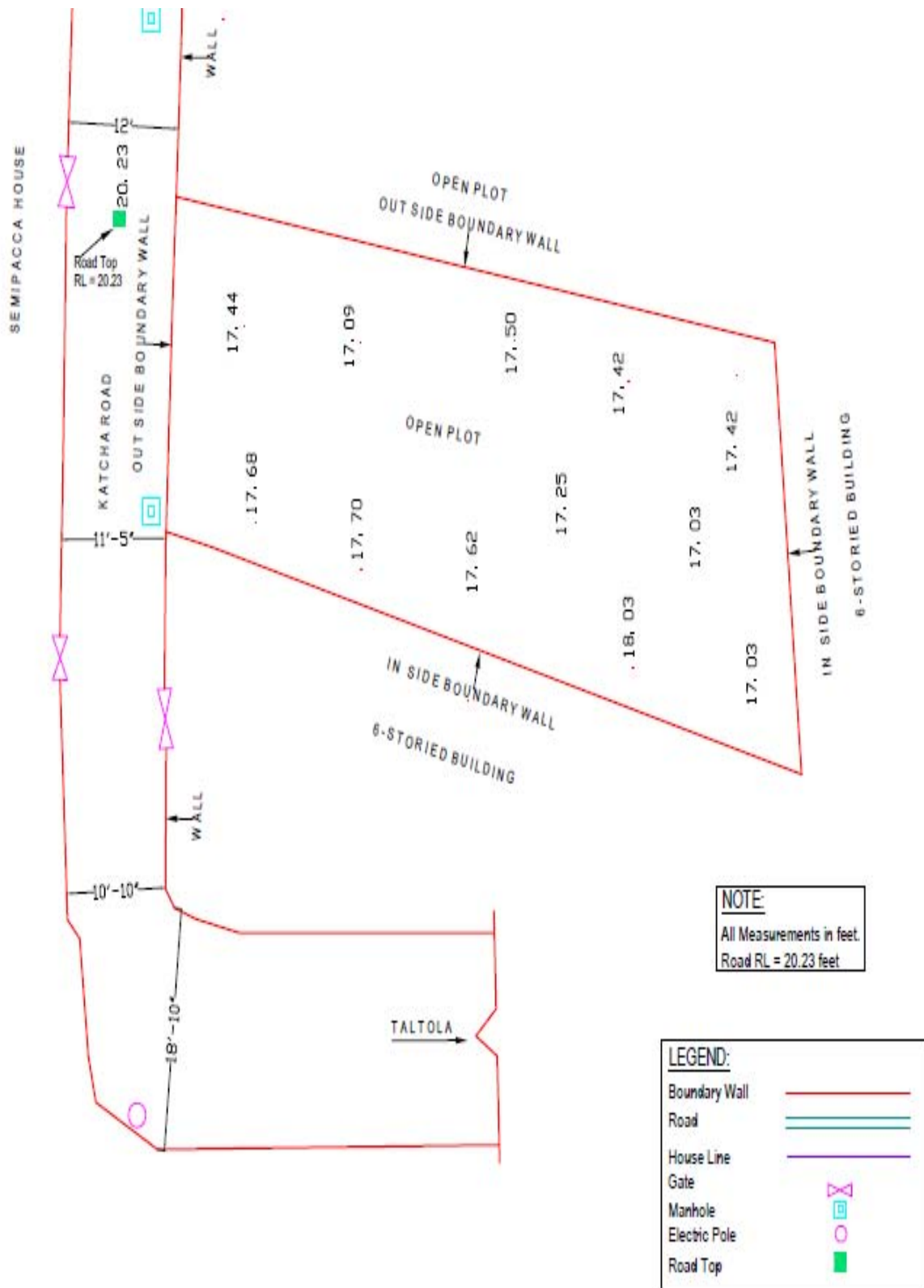
Fig. 2: Topographical map of target highrise site

53
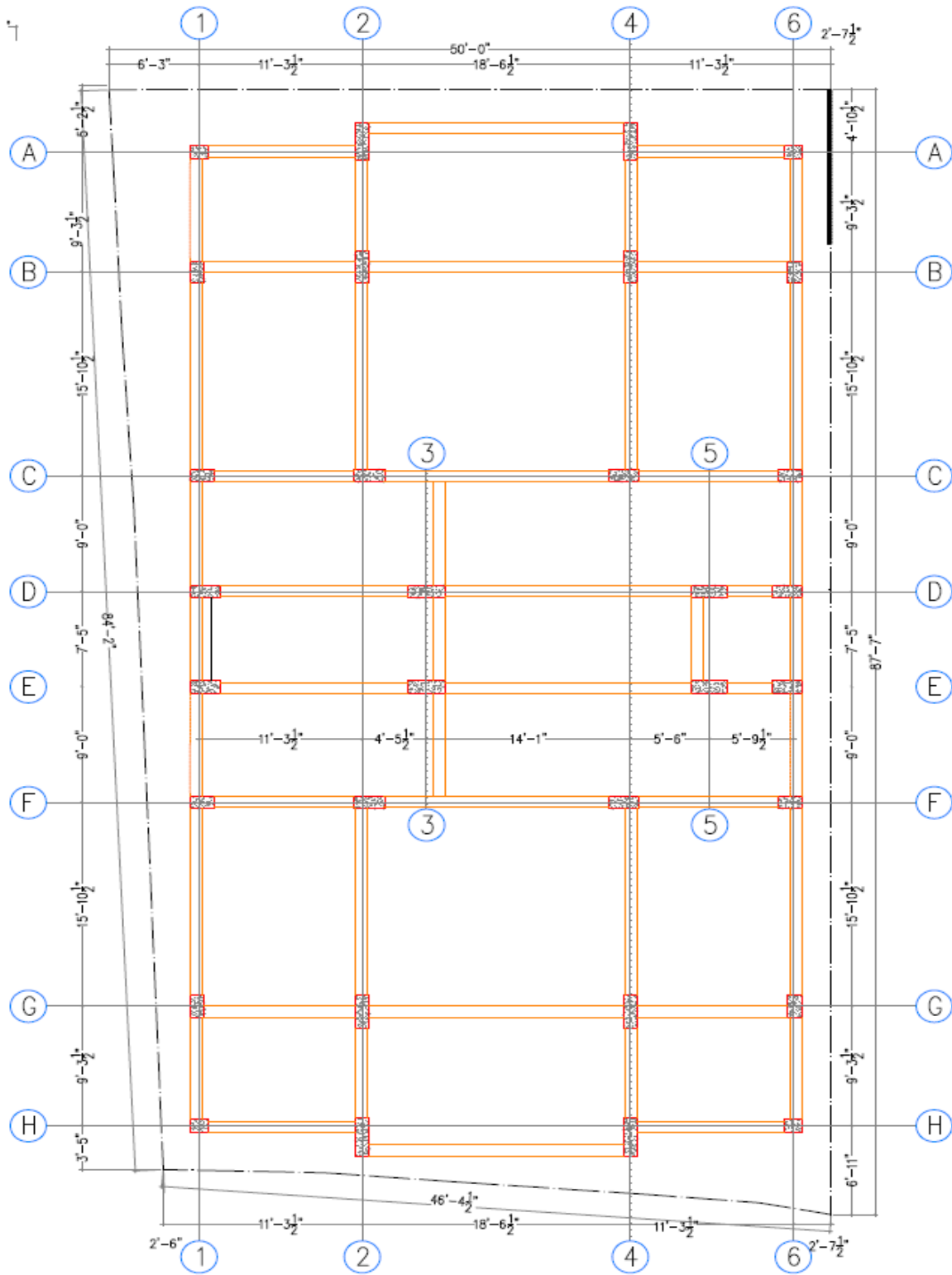
Fig. 3: Structural Drafting of Target Highrise building.

IV. Analysis Process

This target highrise building is modeled on the ETABS 2015 interface using ACI 318-14 design code. It contained a shear wall and several flights of stairs. For simplicity of the analysis no lateral wind load was calculated, so load combination became very simpler, as the frame was simple OMRF.
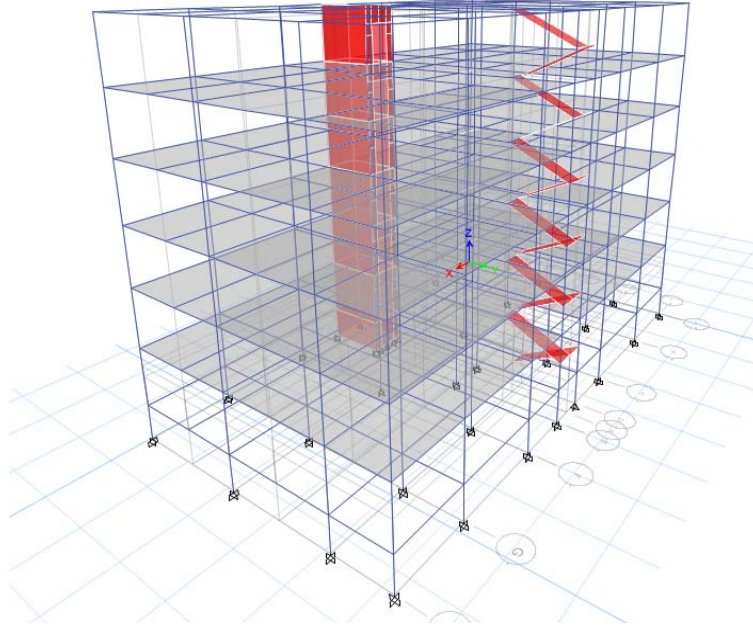
Fig. 4: 3D model of finished building in ETABS 2015 interface.

The beams were 18inches x 18inches in section(4000psi strength), columns were 15inches x 18inches(5000psi strength) and the slab contained a thickness of 6 inches. Shear wall was 7 inches thick. A few conceptual terms are described below to avoid confusion during analysis process.

Capacity: It is defined as the ultimate strength of the structural components excluding the reduction factors commonly used in design of concrete members.

Capacity Curve: Plot between base shear and roof displacement is termed as capacity/pushover curve.

Capacity Spectrum: The capacity curve transformed from base shear vs roof displacement to spectral acceleration vs spectral displacement is termed as capacity spectrum.

Capacity Spectrum method: A nonlinear static procedure that produce a graphical representation of expected seismic performance of building by intersecting capacity curve and response spectrum representation of earthquakes displacement demand on structure, the intersecting point is called performance point.

Demand: It is represented by an estimation of displacement/deformation structure is expected to undergo.

Plastic Hinges: The maximum moments occur near the ends of beams and columns, the plastic hinges are likely to form there and most ductility requirements apply near the section of the junction.

There are mainly four steps for this analysis:
(a) Modeling, (b) Static Analysis, (c) Design, (d) Pushover Analysis

At first the plan was executed on ETABS 2015 interface. Earthquake and Wind forces have been introduced for static loading, and diaphragm has been initiated into the floor plan to pinpoint locations of building stiffness centre. And after the design of concrete moment resisting frame and reinforcement detailing, pushover has been introduced. The building has been shaken in X and Y direction with a maximum target displacement of 31.84 inches and capacity curves have been formed. Afterwards plastic hinges have been formed on each beam span at 0.05 and 0.95 distances(near each end portions)of beams and columns. It is to be noted that active hinge formations are important for development of yield zones in frames.
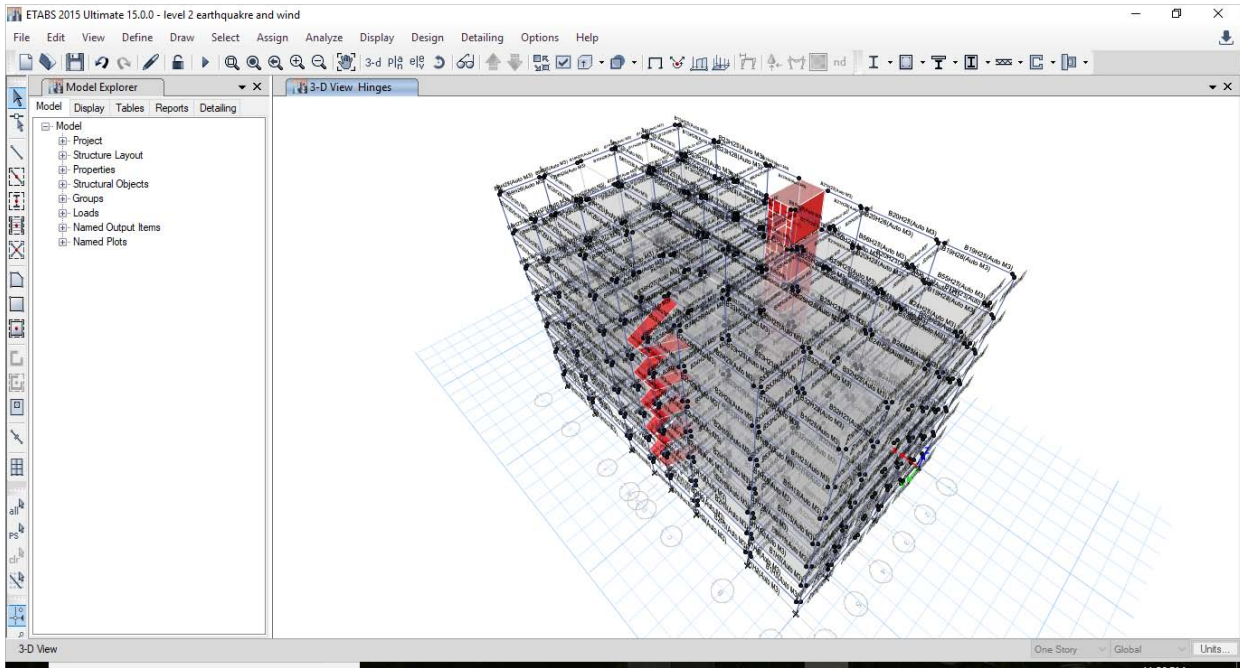
Fig. 5: Plastic hinge formation of target building.

After the analysis pushover curves and hinges are formed. As the target building did not cross the allowable displacement limit, it did not budge from LS(Life safety level-green zone of the pushover curve). Also in PushX and PushY three steps of force-displacement have been generated, showing green hinges, and proving load displacement was in linear static level.

Hinge results and capacity curves formed are below.
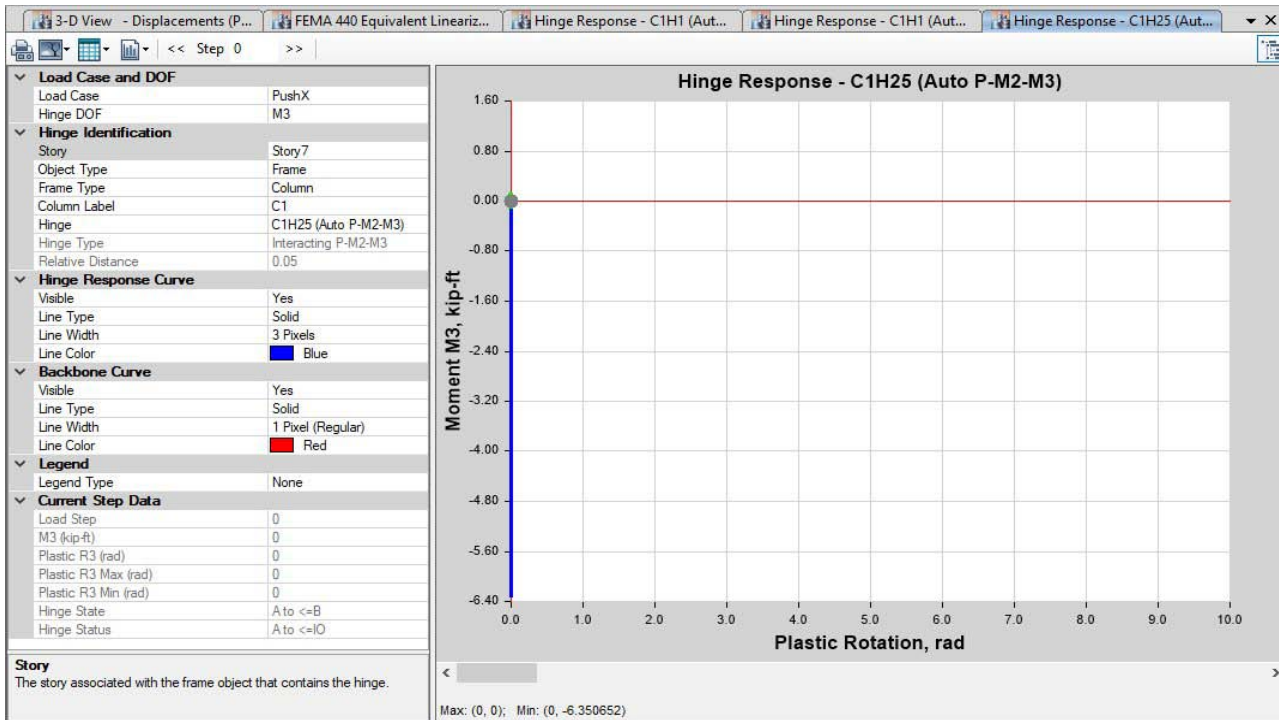
Fig. 6: Hinge response of story 7.

Similar to story 07, all other stories have been seen to be form zero rotation hinges, which subsequently indicate loading was within target level.

Pushover curves have also been formed from FEMA 440 equivalent linearisation process.
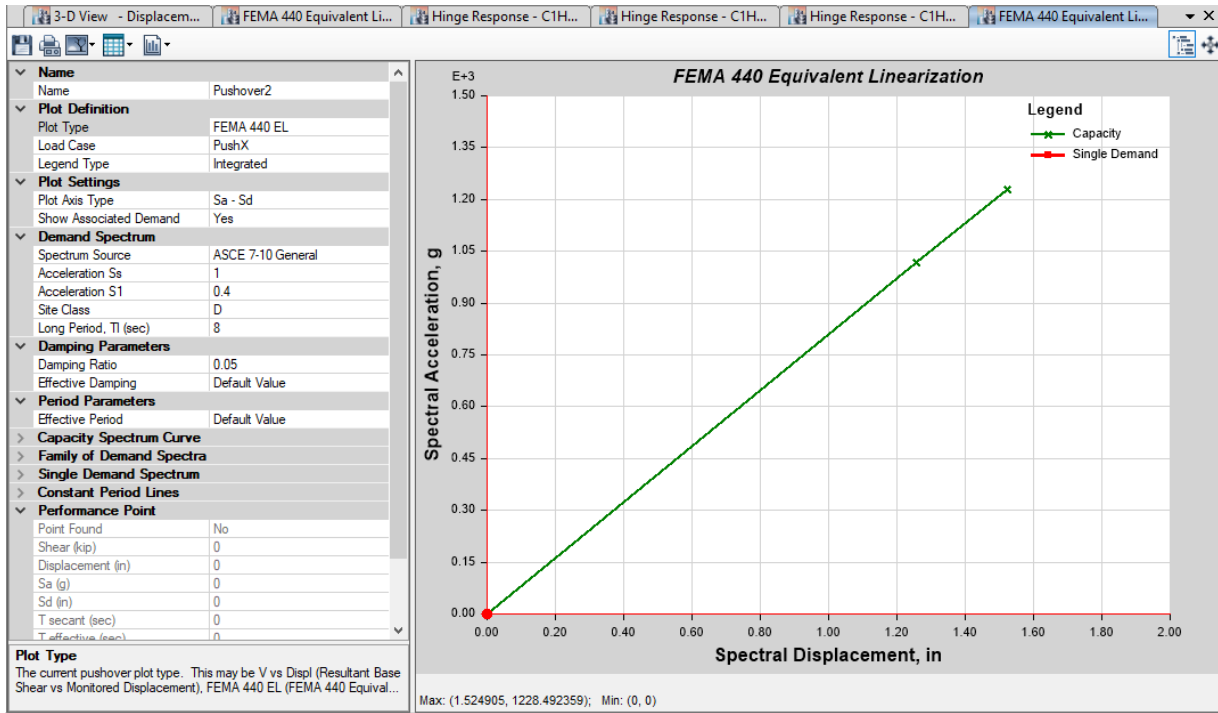
Fig. 7: Pushover capacity curve for Direction X.
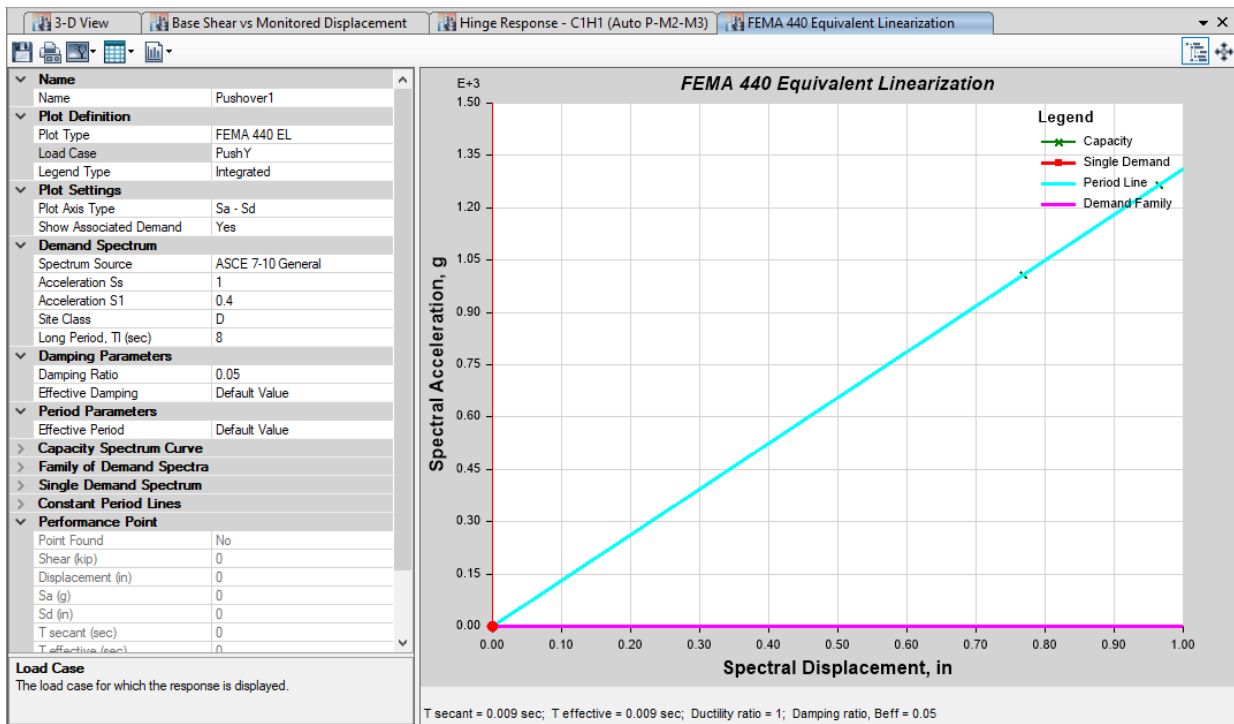
Fig. 8: Pushover capacity curve for Direction Y

V. Scope for Future Studies

This work mainly focused on static pushover of a simplified OMRF frame system which does not contain any kind of seismic detailing, but the study can be further expanded for IMRF and SMRF frames containing steel or composite frame system(framing with bearing walls). Framing systems with irregular plan systems can also be tested by this method. This article focuses on creating two or three steps on push X and Y directions which can be magnified to get a good look on the hinge formation. Finally, critical systems as flat plate slab systems can be tested to examine their behavior under seismic shaking.

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Study the Impact of the Drift (Lateral Deflection) of the Tall Buildings Due to Seismic Load in Concrete Frame Structures with Different Type of RC Shear Walls

By Mahdi Hosseini & N.V. Ramana Rao

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Abstract- Story Drift is defined as the difference in lateral deflection between two adjacent stories. Lateral deflection and drift have three effects on a structure; the movement can affect the structural elements (such as beams and columns); the movements can affect non-structural elements (such as the windows and cladding); and the movements can affect adjacent structures. Without proper consideration during the design process, large deflections and drifts can have adverse effects on structural elements, nonstructural elements, and adjacent structures. Drift problem as the horizontal displacement of all tall buildings is one of the most serious issues in tall building design, relating to the dynamic characteristics of the building during earthquakes and strong winds. Drift shall be caused by the accumulated deformations of each member, such as a beam, column and shear wall. lateral forces due to wind or seismic loading must be considered for tall building design along with gravity forces vertical loads. Tall and slender buildings are strongly wind sensitive and wind forces are applied to the exposed surfaces of the building, whereas seismic forces are inertial (body forces), which result from the distortion of the ground and the inertial resistance of the building.

Keywords: dynamic analysis, seismic load, story drift, RC shear walls, software ETABS.

GJRE-E Classification: FOR Code: 090599

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Keywords: dynamic analysis, seismic load, story drift, RC shear walls, software ETABS.

I. Introduction

a) Earthquake Load

Earthquake forces experienced by a building result from ground motions (accelerations) which are also fluctuating or dynamic in nature, in fact they reverse direction some what chaotically. The magnitude of an earthquake force depends on the magnitude of an earthquake, distance from the earthquake source(epicenter), local ground conditions that may amplify ground shaking (or dampen it), the weight(or mass) of the structure, and the type of structural system and its ability to with stand abusive cyclic loading. In theory and practice, the lateral force that a building experiences from an earthquake increases in direct proportion with the acceleration of ground motion at the building site and the mass of the building (i.e., a doubling in ground motion acceleration or building mass will double the load).This theory rests on the simplicity and validity of Newton's law of physics: F = m x a, where 'F' represents force, 'm' represents mass or weight, and 'a' represents acceleration. For example, as a car accelerates forward, a force is imparted to the

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driver through the seat to push him forward with the car (this force is equivalent to the weight of the driver multiplied by the acceleration or rate of change in speed of the car). As the brake is applied, the car is decelerated and a force is imparted to the driver by the seatbelt to push him back toward the seat. Similarly, as the ground accelerates back and forth during an earthquake, it imparts back-and-forth (cyclic) forces to a building through its foundation which is forced to move to the ground. One can imagine a very light structure such as a fabric tent that will be undamaged in almost any earthquake but it will not survive high wind. The reason is the low mass (weight) of the tent. Therefore, residential buildings generally perform reasonably well in earthquakes, but are more vulnerable in high-wind load prone areas. Regardless, the proper amount of bracing is required in both cases.

Story drift, which is defined here as the relative horizontal displacement of two adjacent floors, can form the starting point for assessment of damage to non-structural components such as facades and interior partitions. However, it is more informative in high-rise buildings to assess these relative movements in each story as components due to:

A) Rigid body displacement.

b) Racking (shear) deformation.

Rigid body displacement is associated with the 'rotation' of the building as a whole at upper levels due to vertical deformations in the columns below, and induces no damage.

Racking hear deformation is a measure of the angular in-plane deformation of a wall or cladding panel. This will in general vary at different positions on a floor, and may exceed the story drift ratio in some locations, (e.g. partition panels spanning between a core and a perimeter column). Inelastic element deformations form the basis for assessment of structural damage and potential for structural collapse. Assessments are generally performed one component at a time by comparing deformation demands with permissible values (e.g., maximum plastic hinge rotations) that are based on structural details (e.g. tie spacing in concrete elements) and co-existing member forces.

When a building is subjected to wind or earthquake load, various types of failure must be prevented:

• Slipping off the foundation (sliding)
• Overturning and uplift (anchorage failure)
• Shear distortion (drift or racking deflection)
• Collapse (excessive racking deflection)

The first three types of failure are schematically shown in the Figure. 1 Clearly, the entire system must be tied together to prevent building collapse or significant deformation.
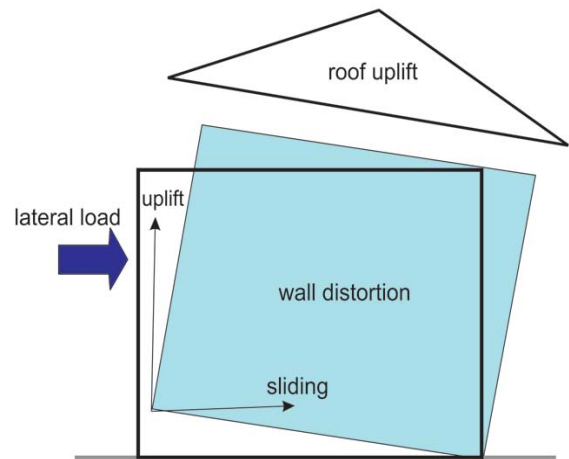
Fig. 1: Schematic of the deformations of the structure due to the lateral loads

II. Methodology

Earthquake motion causes vibration of the structure leading to inertia forces. Thus a structure must be able to safely transmit the horizontal and the vertical inertia forces generated in the super structure through the foundation to the ground. Hence, for most of the ordinary structures, earthquake-resistant design requires ensuring that the structure has adequate lateral load carrying capacity. Seismic codes will guide a designer to safely design the structure for its intended purpose. Seismic codes are unique to a particular region or country. In India, IS 1893(Part1): 2002is the main code that provides outline for calculating seismic design force. This force depends on the mass and seismic coefficient of the structure and the latter in turn depends on properties like seismic zone in which structure lies, importance of the structure, its stiffness, the soil on which it rests, and its ductility. IS 1893 (Part 1): 2002deals with assessment of seismic loads on various structures and buildings. Whole the code centers on the calculation of base shear and its distribution over height.

The analysis can be performed on the basis of the external action, the behavior of the structure or structural materials, and the type of structural model selected. Depending on the height of the structure and zone to which it belongs, type of analysis is performed. In all the methods of analyzing multi- storey buildings recommended in the code, the structure is treated as discrete system having concentrated masses at floor levels, which include half that of columns and walls above and below the floor. In addition, appropriate amount of live load at this floor is also lumped with it.

Quite a few methods are available for the earthquake analysis of buildings; two of them are presented here:

1- Equivalent Static Lateral Force Method (pseudo static method).
2- Dynamic analysis.

   I. Response spectrum method.
   II. Time history method.

a) Dynamic analysis

Dynamic analysis shall be performed to obtain the design seismic force, and its distribution in different levels along the height of the building, and in the various lateral load resisting element, for the following buildings:

Regular buildings: Those greater than 40m in height in zones IV and V, those greater than 90m in height in zone II and III.

Irregular buildings: All framed buildings higher than 12m in zones IV and V, and those greater than 40m in height in zones II and III.

The analysis of model for dynamic analysis of buildings with unusual configuration should be such that it adequately models the types of irregularities present in the building configuration. Buildings with plan irregularities, as defined in Table 4 of IS code: 1893-2002 cannot be modeled for dynamic analysis.

Dynamic analysis may be performed either by the TIME HISTORY METHOD or by the RESPONSE SPECTRUM METHOD

b) Time History Method

The usage of this method shall be on an appropriate ground motion and shall be performed using accepted principles of dynamics. In this method, the mathematical model of the building is subjected to accelerations from earthquake records that represent the expected earthquake at the base of the structure.

c) Response Spectrum Method

The word spectrum in engineering conveys the idea that the response of buildings having a broad range of periods is summarized in a single graph. This method shall be performed using the design spectrum specified in code or by a site-specific design spectrum for a structure prepared at a project site. The values of damping for building may be taken as 2 and 5 percent of the critical, for the purposes of dynamic of steel and reinforce concrete buildings, respectively. For most buildings, inelastic response can be expected to occur during a major earthquake, implying that an inelastic analysis is more proper for design. However, in spite of the availability of nonlinear inelastic programs, they are not used in typical design practice because:

1- Their proper use requires knowledge of their inner workings and theories. design criteria, and

2- Result produced are difficult to interpret and apply to traditional design criteria, and
3- The necessary computations are expensive.

Therefore, analysis in practice typically use linear elastic procedures based on the response spectrum method. The response spectrum analysis is the preferred method because it is easier to use.

d) Response Spectrum Analysis

This method is also known as modal method or mode superposition method. It is based on the idea that the response of a building is the superposition of the responses of individual modes of vibration, each mode responding with its own particular deformed shape, its own frequency, and with its own modal damping.

According to IS-1893(Part-I): 2002, high rise and irregular buildings must be analyzed by response spectrum method using design spectra shown in Figure 4.1. There are significant computational advantages using response spectra method of seismic analysis for prediction of displacements and member forces in structural systems. The method involves only the calculation of the maximum values of the displacements and member forces in each mode using smooth spectra that are the average of several earthquake motions. Sufficient modes to capture such that at least 90% of the participating mass of the building (in each of two orthogonal principle horizontal directions) have to be considered for the analysis. The analysis is performed to determine the base shear for each mode using given building characteristics and ground motion spectra. And then the storey forces, accelerations, and displacements are calculated for each mode, and are combined statistically using the SRSS combination. However, in this method, the design base shear (VB) shall be compared with a base shear (Vb) calculated using a fundamental period T. If V_B is less than V_b response quantities are (for example member forces, displacements, storey forces, storey shears and base reactions) multiplied by VB/V_b Response spectrum method of analysis shall be performed using design spectrum. In case design spectrum is specifically prepared for a structure at a particular project site, the same may be used for design at the discretion of the project authorities. Figure 4.1 shows the proposed 5% spectra for rocky and soils sites.

e) Seismic Analysis Procedure as per the Code

When a structure is subjected to earthquake, it responds by vibrating. An example force can be resolved into three mutually perpendicular directions-two horizontal directions (X and Y directions) and the vertical direction (Z). This motion causes the structure to vibrate or shake in all three directions; the predominant direction of shaking is horizontal. All the structures are primarily designed for gravity loads-force equal to mass

time's gravity in the vertical direction. Because of the inherent factor used in the design specifications, most structures tend to be adequately protected against vertical shaking. Vertical acceleration should also be considered in structures with large spans those in which stability for design, or for overall stability analysis of structures. The basic intent of design theory for earthquake resistant structures is that buildings should be able to resist minor earthquakes without damage, resist moderate earthquakes without structural damage but with some non-structural damage. To avoid collapse during a major earthquake, Members must be ductile enough to absorb and dissipate energy by post elastic deformation. Redundancy in the structural system permits redistribution of internal forces in the event of the failure of key elements. When the primary element or system yields or fails, the lateral force can be redistributed to a secondary system to prevent progressive failure.

IS 1893 (part- 1) Code recommends that detailed dynamic analysis, or pseudo static analysis should be carries out depending on the importance of the problems.

IS 1893 (part- 1) Recommends use of model analysis using response spectrum method and equivalent lateral force method for building of height less than 40m in all seismic zones as safe., but practically there may be the building which are more than 40m in height. So there exist so many problems due to the increase in height of the structure.

The earthquake resistant structures are constructed using IS 1893 part-1 and there are some assumptions to be made in the design according to the codal provisions and these assumptions account to one of the uncertainties that occur in the design starting from mix design to workmanship and many other.

The following assumptions shall be made in the earthquake resistant design of structures:

Earthquake causes impulsive ground motions, which are complex and irregular in character, changing in period and amplitude each lasting for a small duration. Therefore, resonance of the type as visualized under steady-state sinusoidal excitations will not occur as it would need time to buildup such amplitudes.

III. Modeling of Building

a) Details of The Building

A symmetrical building of plan 38.5m X 35.5m located with location in zone V, India is considered. Four bays of length 7.5m& one bays of length 8.5m along X - direction and Four bays of length 7.5m& one bays of length 5.5m along Y - direction are provided. Shear Wall is provided at the center core of building model.

Structure 1: In this model building with 30 storey is modeled as a (Dual frame system with shear wall (Plus Shape) at the center of building, The shear wall acts as vertical cantilever.

Structure 2: In this model building with 30 storey is modeled as (Dual frame system with shear wall (Box Shape) at the center of building ,The shear wall acts as vertical cantilever.

Structure 3: In this model building with 30 storey is modeled as (Dual frame system with shear wall (C- Shape) at the center of building, The shear wall acts as vertical cantilever.

Structure 4: In this model building with 30 storey is modeled as (Dual frame system with shear wall (E- Shape) at the center of building ,The shear wall acts as vertical cantilever.

Structure 5: In this model building with 30 storey is modeled as (Dual frame system with shear wall (I - Shape) at the center of building, The shear wall acts as vertical cantilever.

b) Load Combinations

As per IS 1893 (Part 1): 2002 Clause no. 6.3.1.2, the following load cases have to be considered for analysis:

1.5 (DL + IL)

1.2 (DL + IL ± EL)

1.5 (DL ± EL)

0.9 DL ± 1.5 EL

Earthquake load must be considered for +X, -X, +Y and –Y directions.

Table 1: Details of the Building

Building Parameters	Details
Type of frame	Special RC moment resisting frame fixed at the base
Building plan	38.5m X 35.5m
Number of storeys	30
Floor height	3.5 m
Depth of Slab	225 mm
Size of beam	(300×600) mm
Size of column (exterior)	(1250×1250) mm up to story five
Size of column (exterior)	(900×900) mm Above story five

Size of column (interior)	(1250×1250) mm up to story ten
Size of column (interior)	(900×900) mm Above story ten
Spacing between frames	7.5-8.5 m along x - direction 7.5-5.5 m along y - direction
Live load on floor	4 KN/m2
Floor finish	2.5 KN/m2
Wall load	25 KN/m
Grade of Concrete	M 50 concrete
Grade of Steel	Fe 500
Thickness of shear wall	450 mm
Seismic zone	V
Important Factor	1.5
Density of concrete	25 KN/m3
Type of soil	Soft, Medium, Hard Soil Type I=Soft Soil Soil Type II=Medium Soil Soil Type III= Hard Soil
Response spectra	As per IS 1893(Part-1): 2002
Damping of structure	5 percent
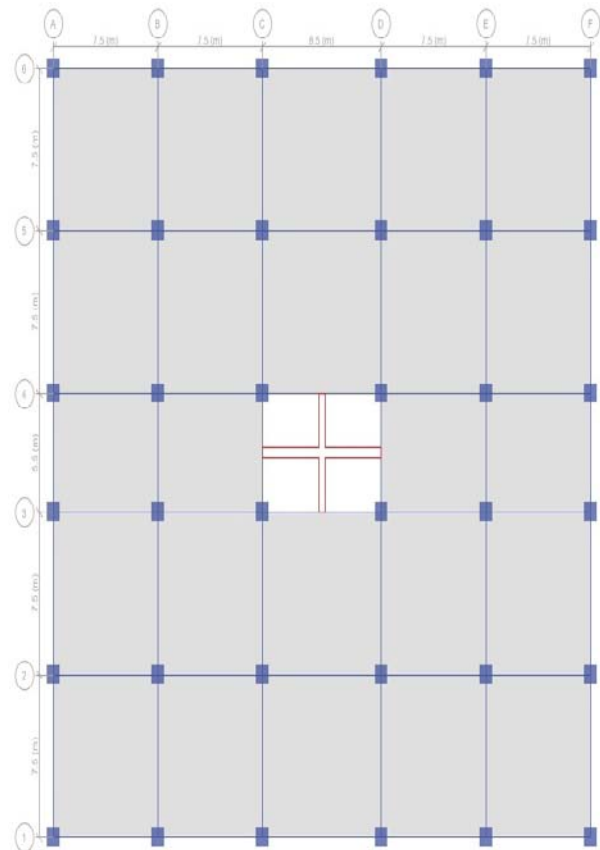
Figure 1: Plan of the Structure 1
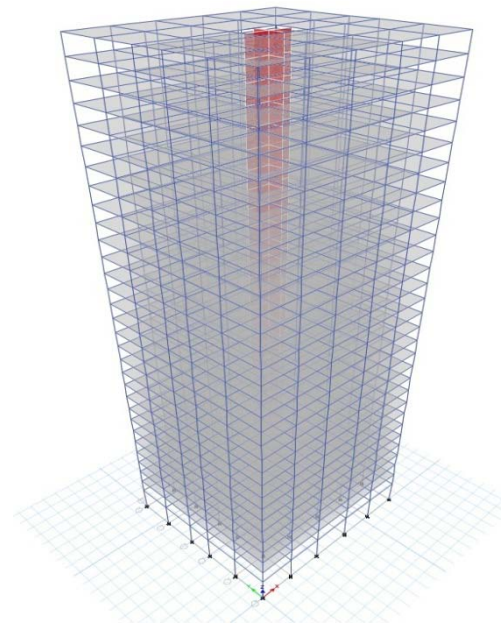
Figure 2: 3D view showing shear wall location for Structure 1
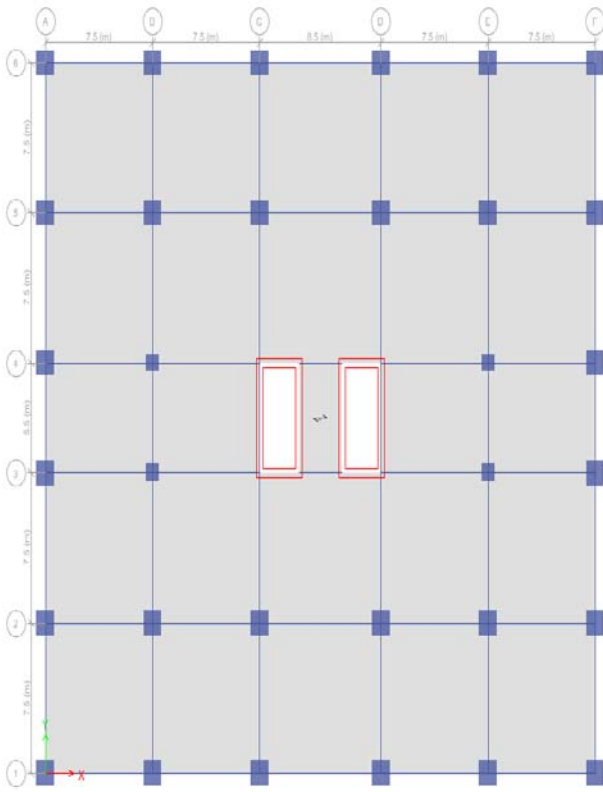
Figure 3: Plan of the Structure 2

Figure 5: Plan of the Structure 3

Figure 4: 3D view showing shear wall location for Structure2
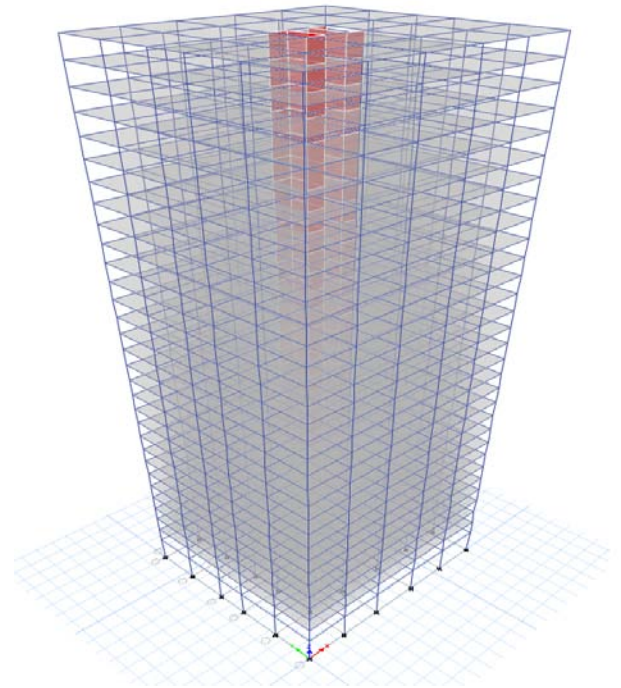
Figure 6: 3D view showing shear wall location for Structure 3
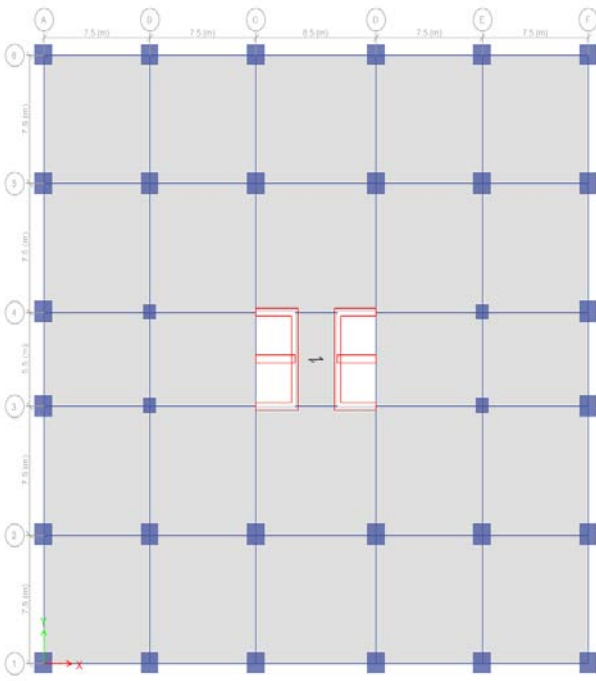
Figure 7: Plan of the Structure 4
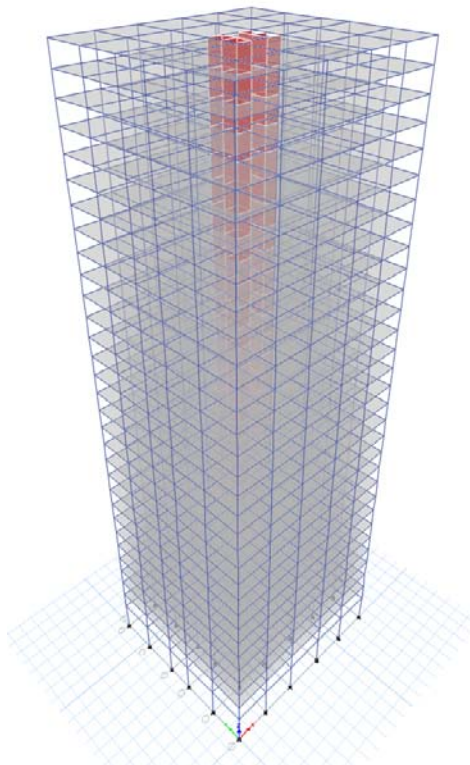
Figure 8: 3D view showing shear wall location for Structure 4
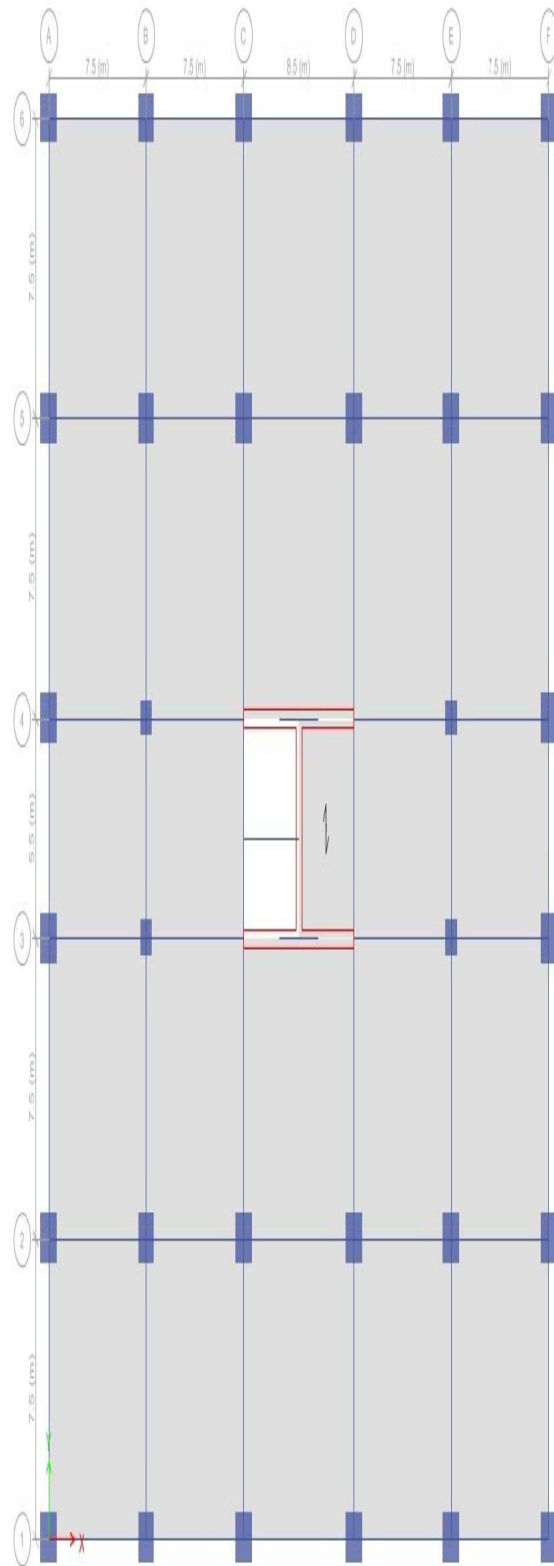
Figure 9: Plan of the Structure 5
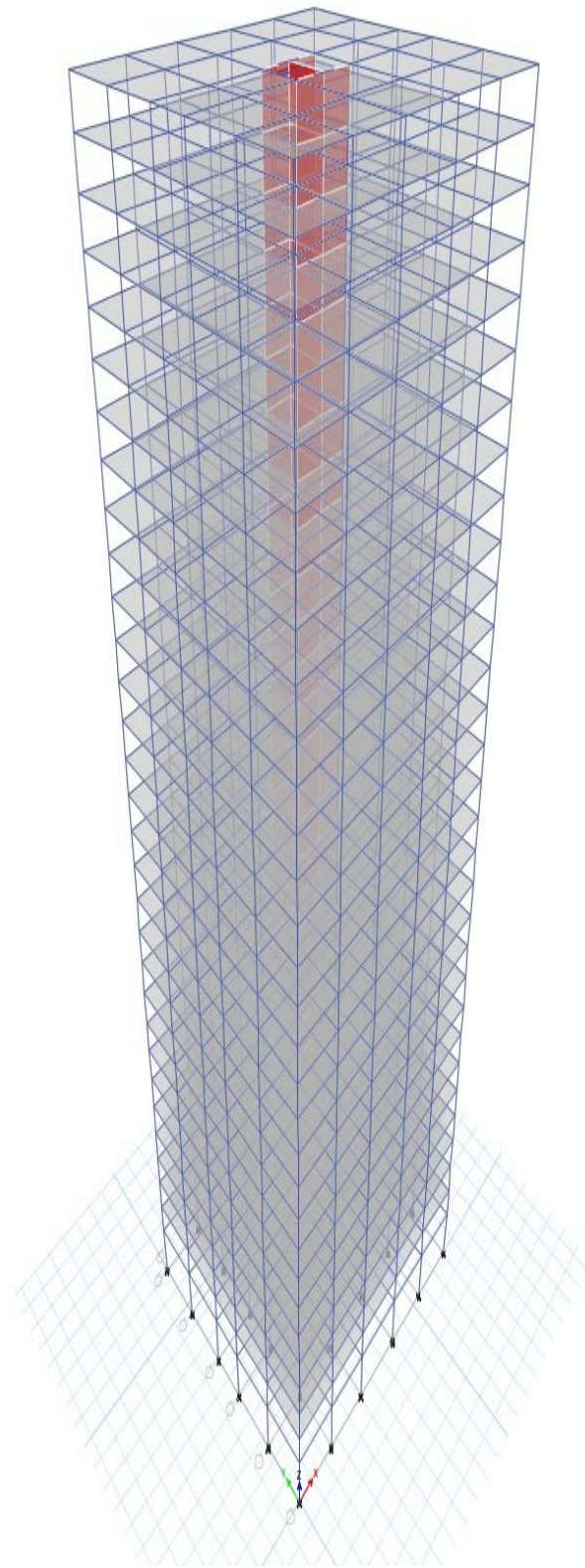
Figure 10: 3D view showing shear wall location for Structure 5

IV. Results and Discussions

Table 2: Storey Drifts of Structures in Soft Soil in X - Direction with load combination (DL+LL+EQXP)

Story	Elevation	Location	Structure -1 X-Dir	Structure -2 X-Dir	Structure -3 X-Dir	Structure -4 X-Dir	Structure -5 X-Dir
	m						
30TH	111	Top	0.001515	0.001454	0.001295	0.001275	0.001889
29TH	107.5	Top	0.001625	0.001533	0.001411	0.001374	0.001959
28TH	104	Top	0.001711	0.001624	0.001562	0.001503	0.002036
27TH	100.5	Top	0.001814	0.001733	0.001736	0.001654	0.002129
26TH	97	Top	0.001925	0.001852	0.001921	0.001816	0.002231
25TH	93.5	Top	0.00204	0.001975	0.002109	0.001981	0.002337
24TH	90	Top	0.002153	0.002098	0.002294	0.002145	0.002445
23RD	86.5	Top	0.002263	0.002219	0.002473	0.002304	0.002552
22ND	83	Top	0.002369	0.002336	0.002643	0.002455	0.002654
21ST	79.5	Top	0.002467	0.002446	0.002802	0.002597	0.002751
20TH	76	Top	0.002557	0.002548	0.002949	0.002729	0.002841
19TH	72.5	Top	0.002638	0.002641	0.003083	0.002849	0.002921
18TH	69	Top	0.002708	0.002724	0.003204	0.002957	0.002991
17TH	65.5	Top	0.002768	0.002796	0.003311	0.003053	0.003049
16TH	62	Top	0.002817	0.002855	0.003403	0.003135	0.003095
15TH	58.5	Top	0.002853	0.002902	0.003481	0.003203	0.003126
14TH	55	Top	0.002876	0.002936	0.003544	0.003258	0.003141
13TH	51.5	Top	0.002885	0.002955	0.00359	0.003296	0.00314
12TH	48	Top	0.002879	0.002957	0.003618	0.003318	0.00312
11TH	44.5	Top	0.002858	0.002944	0.003627	0.003321	0.003081
10TH	41	Top	0.002803	0.002902	0.0036	0.003293	0.003011
9TH	37.5	Top	0.002749	0.002852	0.003566	0.003255	0.00293
8TH	34	Top	0.002668	0.002776	0.003503	0.003191	0.002822
7TH	30.5	Top	0.002562	0.002674	0.003405	0.003094	0.002685
6TH	27	Top	0.00242	0.002537	0.003259	0.002954	0.002511
5TH	23.5	Top	0.00223	0.002354	0.003055	0.002762	0.002294
4TH	20	Top	0.002056	0.00217	0.002853	0.002569	0.00208
3RD	16.5	Top	0.001827	0.001931	0.002578	0.002311	0.001817
2ND	13	Top	0.001548	0.001634	0.002214	0.001974	0.001506
1ST	9.5	Top	0.00122	0.001277	0.001738	0.001539	0.001151
PLINTH	6	Top	0.00056	0.000581	0.000794	0.000698	0.000513
Base	0	Top	0	0	0	0	0

A plot for Storey Drifts of Structures in Soft Soil in X - Direction with load combination (DL+LL+EQXP) has been shown here
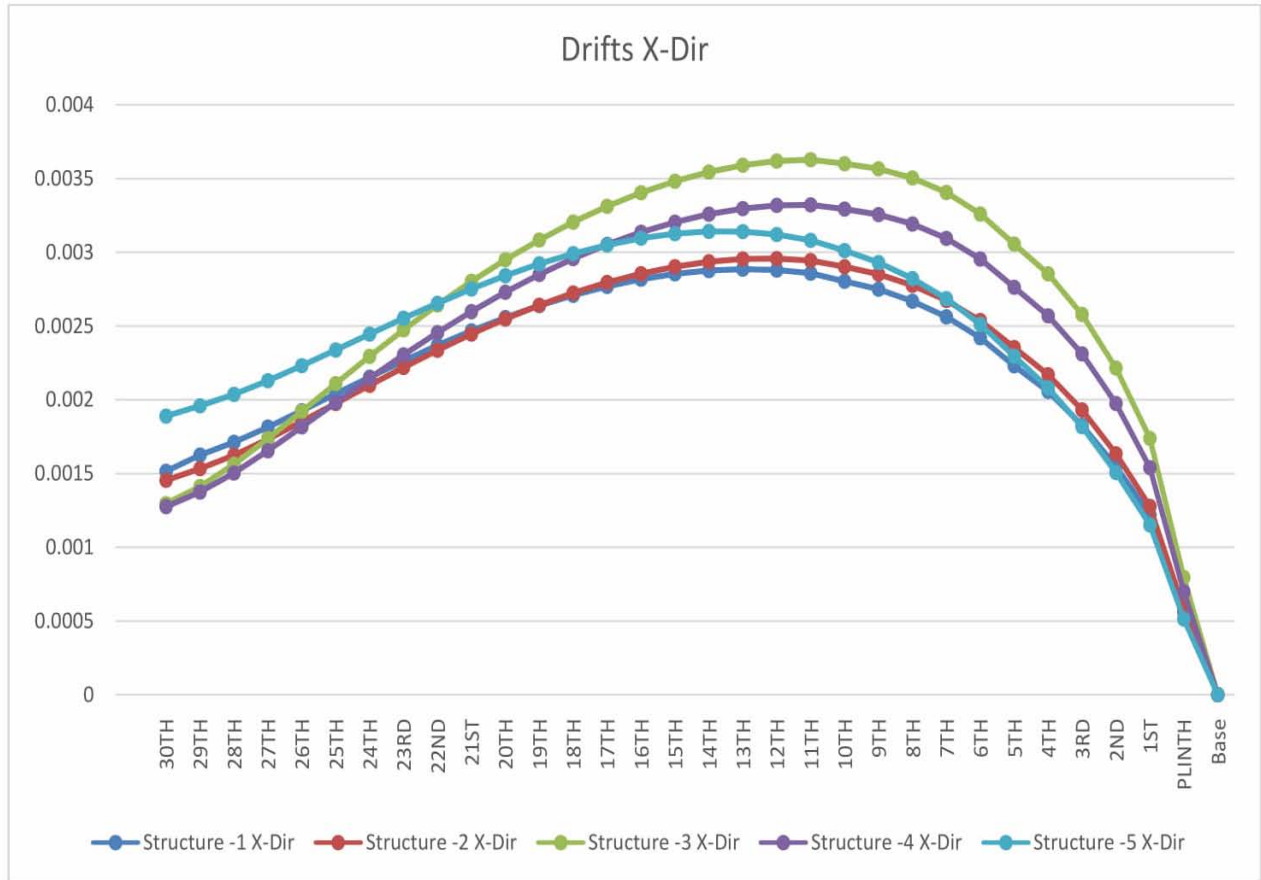
Graph 1: Storey Drifts of Structures in Soft Soil in X - Direction

Table 3: Storey Drifts of Structures in Medium Soil in X - Direction with load combination (DL+LL+EQXP)

Story	Elevation	Location	Structure -1 X-Dir	Structure -2 X-Dir	Structure -3 X-Dir	Structure -4 X-Dir	Structure -5 X-Dir
	m						
30TH	111	Top	0.002059	0.001977	0.001761	0.001843	0.002552
29TH	107.5	Top	0.002208	0.002085	0.00192	0.001985	0.002647
28TH	104	Top	0.002326	0.002209	0.002124	0.002172	0.002752
27TH	100.5	Top	0.002465	0.002357	0.00236	0.00239	0.002878
26TH	97	Top	0.002617	0.002518	0.002612	0.002623	0.003017
25TH	93.5	Top	0.002772	0.002685	0.002868	0.002861	0.003162
24TH	90	Top	0.002927	0.002853	0.00312	0.003098	0.003309
23RD	86.5	Top	0.003077	0.003018	0.003364	0.003327	0.003454
22ND	83	Top	0.00322	0.003177	0.003595	0.003546	0.003594
21ST	79.5	Top	0.003353	0.003327	0.003811	0.003751	0.003726
20TH	76	Top	0.003476	0.003466	0.004011	0.003941	0.003848
19TH	72.5	Top	0.003586	0.003592	0.004193	0.004115	0.003957

18TH	69	Top	0.003682	0.003705	0.004357	0.004271	0.004053
17TH	65.5	Top	0.003764	0.003802	0.004503	0.004408	0.004132
16TH	62	Top	0.00383	0.003883	0.004628	0.004527	0.004194
15TH	58.5	Top	0.003879	0.003947	0.004734	0.004626	0.004237
14TH	55	Top	0.003911	0.003993	0.004819	0.004704	0.004258
13TH	51.5	Top	0.003923	0.004018	0.004882	0.00476	0.004257
12TH	48	Top	0.003914	0.004022	0.00492	0.004791	0.00423
11TH	44.5	Top	0.003885	0.004004	0.004933	0.004795	0.004177
10TH	41	Top	0.003812	0.003947	0.004897	0.004755	0.004083
9TH	37.5	Top	0.003737	0.003879	0.00485	0.0047	0.003974
8TH	34	Top	0.003627	0.003776	0.004764	0.004607	0.003828
7TH	30.5	Top	0.003483	0.003637	0.004631	0.004467	0.003643
6TH	27	Top	0.00329	0.003451	0.004433	0.004265	0.003407
5TH	23.5	Top	0.003032	0.003202	0.004154	0.003988	0.003112
4TH	20	Top	0.002795	0.002952	0.00388	0.003709	0.002823
3RD	16.5	Top	0.002485	0.002627	0.003507	0.003337	0.002466
2ND	13	Top	0.002104	0.002223	0.003011	0.002851	0.002045
1ST	9.5	Top	0.001656	0.001733	0.00236	0.002218	0.001561
PLINTH	6	Top	0.00076	0.000787	0.001079	0.001006	0.000695
Base	0	Top	0	0	0	0	0

A plot for Storey Drifts of Structures in Medium Soil in X - Direction with load combination (DL+LL+EQXP) has been shown here

Graph 2: Storey Drifts of Structures in Medium Soil in X - Direction

Table 4: Storey Drifts of Structures in Hard Soil in X - Direction with load combination (DL+LL+EQXP)

Story	Elevation	Location	Structure -1 X-Dir	Structure -2 X-Dir	Structure -3 X-Dir	Structure -4 X-Dir	Structure -5 X-Dir
	m						
30TH	111	Top	0.002527	0.002428	0.002163	0.002263	0.003123
29TH	107.5	Top	0.002711	0.00256	0.002357	0.002438	0.00324
28TH	104	Top	0.002855	0.002713	0.002608	0.002667	0.003369
27TH	100.5	Top	0.003026	0.002894	0.002899	0.002935	0.003524
26TH	97	Top	0.003213	0.003092	0.003208	0.003221	0.003694
25TH	93.5	Top	0.003403	0.003297	0.003522	0.003514	0.003872
24TH	90	Top	0.003593	0.003504	0.003832	0.003804	0.004052
23RD	86.5	Top	0.003777	0.003706	0.004131	0.004086	0.004231
22ND	83	Top	0.003953	0.003901	0.004414	0.004354	0.004402
21ST	79.5	Top	0.004117	0.004085	0.00468	0.004606	0.004565
20TH	76	Top	0.004267	0.004256	0.004925	0.00484	0.004715
19TH	72.5	Top	0.004402	0.004411	0.005149	0.005053	0.004849
18TH	69	Top	0.004521	0.004549	0.005351	0.005244	0.004967
17TH	65.5	Top	0.004621	0.004669	0.005529	0.005413	0.005064
16TH	62	Top	0.004702	0.004768	0.005684	0.005559	0.005141
15TH	58.5	Top	0.004762	0.004847	0.005814	0.00568	0.005193
14TH	55	Top	0.004801	0.004903	0.005918	0.005776	0.00522
13TH	51.5	Top	0.004816	0.004934	0.005995	0.005845	0.005219
12TH	48	Top	0.004805	0.004939	0.006042	0.005883	0.005186
11TH	44.5	Top	0.00477	0.004917	0.006057	0.005888	0.005122
10TH	41	Top	0.00468	0.004847	0.006013	0.005838	0.005006
9TH	37.5	Top	0.004588	0.004763	0.005956	0.005771	0.004873
8TH	34	Top	0.004454	0.004637	0.00585	0.005657	0.004694
7TH	30.5	Top	0.004276	0.004466	0.005686	0.005485	0.004467
6TH	27	Top	0.004039	0.004237	0.005443	0.005237	0.004179
5TH	23.5	Top	0.003723	0.003932	0.005101	0.004897	0.003817
4TH	20	Top	0.003432	0.003625	0.004764	0.004554	0.003463
3RD	16.5	Top	0.003051	0.003225	0.004306	0.004097	0.003026
2ND	13	Top	0.002584	0.002729	0.003698	0.0035	0.002509
1ST	9.5	Top	0.002031	0.002126	0.002896	0.002721	0.001913
PLINTH	6	Top	0.000932	0.000967	0.001324	0.001235	0.000852
Base	0	Top	0	0	0	0	0

A plot for Storey Drifts of Structures in Hard Soil in X - Direction with load combination (DL+LL+EQXP) has been shown here

Graph 3: Storey Drifts of Structures in Hard Soil in X - Direction

Table 5: Storey Drifts of Structures in Soft Soil in Y - Direction with load combination (DL+LL+EQYP)

			Structure -1	Structure -2	Structure -3	Structure -4	Structure -5
Story	Elevation	Location	Y-Dir	Y-Dir	Y-Dir	Y-Dir	Y-Dir
	m						
30TH	111	Top	0.001312	0.002009	0.00198	0.00188	0.00197
29TH	107.5	Top	0.001449	0.002063	0.002063	0.001959	0.002069
28TH	104	Top	0.001599	0.002121	0.002146	0.002037	0.002152
27TH	100.5	Top	0.001773	0.002192	0.002243	0.002129	0.002243
26TH	97	Top	0.001958	0.002269	0.002347	0.002228	0.00234
25TH	93.5	Top	0.002145	0.002349	0.002455	0.00233	0.002439
24TH	90	Top	0.002329	0.002431	0.002563	0.002433	0.002536
23RD	86.5	Top	0.002505	0.002511	0.002669	0.002533	0.002631
22ND	83	Top	0.002672	0.002587	0.002771	0.002629	0.002721
21ST	79.5	Top	0.002828	0.002659	0.002866	0.002719	0.002804
20TH	76	Top	0.002971	0.002723	0.002953	0.002802	0.002879
19TH	72.5	Top	0.003102	0.00278	0.00303	0.002875	0.002945
18TH	69	Top	0.003218	0.002827	0.003097	0.002939	0.003001
17TH	65.5	Top	0.003321	0.002864	0.003152	0.00299	0.003045
16TH	62	Top	0.00341	0.002888	0.003193	0.00303	0.003076
15TH	58.5	Top	0.003483	0.0029	0.00322	0.003055	0.003094
14TH	55	Top	0.003542	0.002898	0.003232	0.003066	0.003098
13TH	51.5	Top	0.003584	0.002881	0.003226	0.003061	0.003086
12TH	48	Top	0.003608	0.002848	0.003202	0.003038	0.003057
11TH	44.5	Top	0.003615	0.002798	0.003159	0.002997	0.00301
10TH	41	Top	0.003579	0.002723	0.003081	0.002924	0.002928
9TH	37.5	Top	0.003544	0.002637	0.002998	0.002845	0.002848
8TH	34	Top	0.003478	0.002528	0.002887	0.002739	0.00274
7TH	30.5	Top	0.003379	0.002396	0.002747	0.002606	0.002606
6TH	27	Top	0.003232	0.002236	0.002568	0.002437	0.002435
5TH	23.5	Top	0.003021	0.002039	0.002343	0.002224	0.00222
4TH	20	Top	0.002827	0.00184	0.002133	0.002024	0.002032
3RD	16.5	Top	0.002559	0.001602	0.001874	0.001778	0.001797
2ND	13	Top	0.002202	0.001325	0.001568	0.001487	0.001523
1ST	9.5	Top	0.001748	0.001011	0.001217	0.001155	0.001213
PLINTH	6	Top	0.00081	0.00045	0.000551	0.000523	0.000564
Base	0	Top	0	0	0	0	0

A plot for Storey Drifts of Structures in Soft Soil in Y - Direction with load combination (DL+LL+EQXP) has been shown here
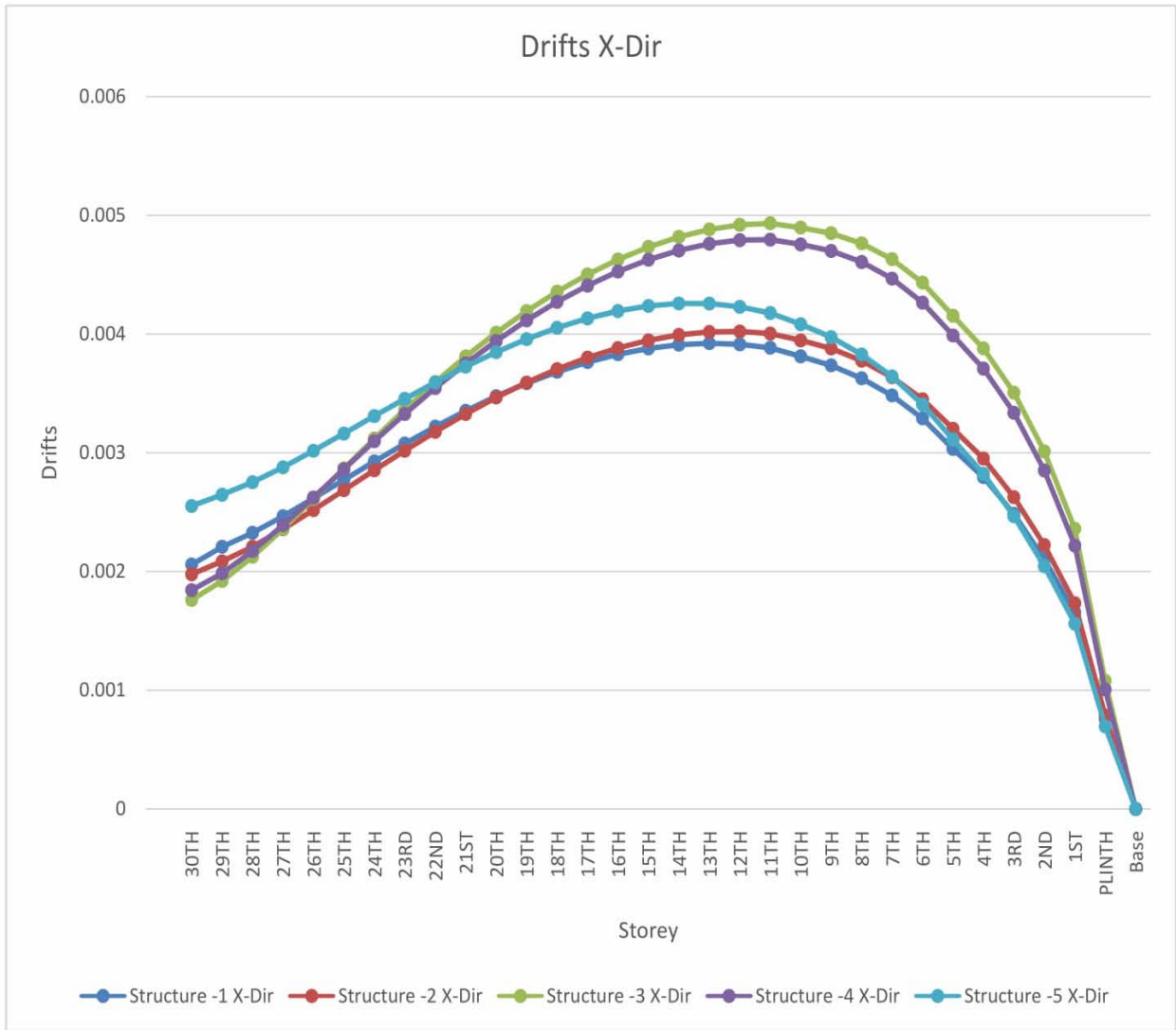

Graph 4: Storey Drifts of Structures in Soft Soil in Y - Direction

Table 6: Storey Drifts of Structures in Medium Soil in Y - Direction with load combination (DL+LL+EQYP)

			Structure -1	Structure -2	Structure -3	Structure -4	Structure -5
Story	Elevation	Location	Y-Dir	Y-Dir	Y-Dir	Y-Dir	Y-Dir
	m						
30TH	111	Top	0.001791	0.002732	0.002693	0.002717	0.002681
29TH	107.5	Top	0.001977	0.002806	0.002806	0.00283	0.002816
28TH	104	Top	0.002182	0.002885	0.002918	0.002942	0.002928
27TH	100.5	Top	0.002419	0.002981	0.00305	0.003075	0.003053
26TH	97	Top	0.002671	0.003085	0.003192	0.003218	0.003184
25TH	93.5	Top	0.002926	0.003195	0.003339	0.003365	0.003318

24TH	90	Top	0.003175	0.003306	0.003486	0.003514	0.003451
23RD	86.5	Top	0.003415	0.003414	0.00363	0.003659	0.00358
22ND	83	Top	0.003642	0.003518	0.003768	0.003797	0.003702
21ST	79.5	Top	0.003854	0.003616	0.003897	0.003927	0.003815
20TH	76	Top	0.004049	0.003704	0.004016	0.004046	0.003917
19TH	72.5	Top	0.004226	0.003781	0.004121	0.004153	0.004007
18TH	69	Top	0.004385	0.003845	0.004212	0.004244	0.004082
17TH	65.5	Top	0.004524	0.003894	0.004286	0.004319	0.004142
16TH	62	Top	0.004645	0.003928	0.004342	0.004375	0.004185
15TH	58.5	Top	0.004745	0.003945	0.004379	0.004412	0.00421
14TH	55	Top	0.004824	0.003942	0.004395	0.004428	0.004215
13TH	51.5	Top	0.004882	0.003919	0.004387	0.00442	0.004198
12TH	48	Top	0.004914	0.003873	0.004355	0.004387	0.004158
11TH	44.5	Top	0.004923	0.003806	0.004296	0.004328	0.004095
10TH	41	Top	0.004874	0.003703	0.00419	0.004222	0.003983
9TH	37.5	Top	0.004826	0.003587	0.004077	0.004108	0.003874
8TH	34	Top	0.004737	0.003438	0.003926	0.003955	0.003728
7TH	30.5	Top	0.004601	0.003259	0.003736	0.003764	0.003546
6TH	27	Top	0.004401	0.00304	0.003493	0.003519	0.003313
5TH	23.5	Top	0.004113	0.002773	0.003187	0.003211	0.003021
4TH	20	Top	0.003849	0.002503	0.002901	0.002923	0.002764
3RD	16.5	Top	0.003484	0.002178	0.002548	0.002567	0.002445
2ND	13	Top	0.002998	0.001801	0.002132	0.002147	0.002072
1ST	9.5	Top	0.002376	0.001372	0.001652	0.001663	0.001647
PLINTH	6	Top	0.001102	0.00061	0.000749	0.000754	0.000766
Base	0	Top	0	0	0	0	0

A plot for Storey Drifts of Structures in Medium Soil in Y - Direction with load combination (DL+LL+EQYP) has been shown here
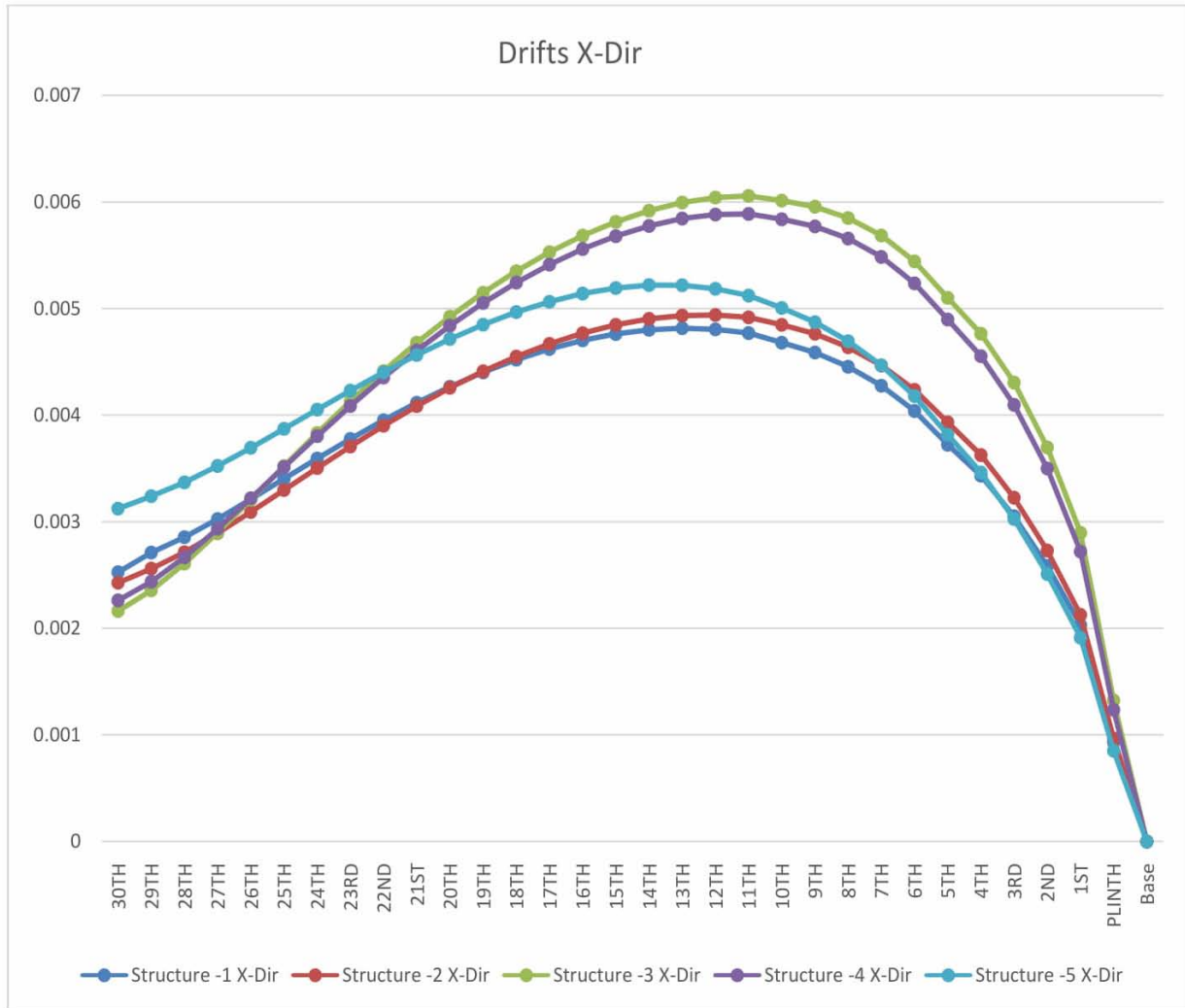
Graph 5: Storey Drifts of Structures in Medium Soil in Y - Direction

Table 7: Storey Drifts of Structures in Hard Soil in Y - Direction with load combination (DL+LL+EQYP)

Story	Elevation	Location	Structure -1 Y-Dir	Structure -2 Y-Dir	Structure -3 Y-Dir	Structure -4 Y-Dir	Structure -5 Y-Dir
	m						
30TH	111	Top	0.002203	0.003354	0.003306	0.003336	0.003293
29TH	107.5	Top	0.002433	0.003446	0.003446	0.003475	0.003459
28TH	104	Top	0.002683	0.003542	0.003584	0.003613	0.003597
27TH	100.5	Top	0.002976	0.00366	0.003746	0.003776	0.00375
26TH	97	Top	0.003285	0.003789	0.00392	0.003951	0.003911
25TH	93.5	Top	0.003597	0.003923	0.0041	0.004133	0.004076
24TH	90	Top	0.003904	0.004059	0.004281	0.004314	0.004239
23RD	86.5	Top	0.004199	0.004193	0.004458	0.004492	0.004397
22ND	83	Top	0.004477	0.00432	0.004627	0.004663	0.004547
21ST	79.5	Top	0.004737	0.00444	0.004786	0.004823	0.004686
20TH	76	Top	0.004977	0.004548	0.004931	0.004969	0.004811
19TH	72.5	Top	0.005194	0.004642	0.00506	0.005099	0.004921
18TH	69	Top	0.005389	0.004721	0.005172	0.005211	0.005014
17TH	65.5	Top	0.005561	0.004782	0.005263	0.005303	0.005088
16TH	62	Top	0.005708	0.004824	0.005332	0.005373	0.00514
15TH	58.5	Top	0.005831	0.004844	0.005378	0.005418	0.00517
14TH	55	Top	0.005929	0.00484	0.005397	0.005437	0.005176
13TH	51.5	Top	0.005999	0.004812	0.005388	0.005428	0.005156
12TH	48	Top	0.006038	0.004756	0.005347	0.005387	0.005107
11TH	44.5	Top	0.00605	0.004673	0.005275	0.005314	0.005029
10TH	41	Top	0.005989	0.004547	0.005145	0.005184	0.004892
9TH	37.5	Top	0.00593	0.004404	0.005007	0.005044	0.004758
8TH	34	Top	0.005821	0.004222	0.004821	0.004857	0.004578
7TH	30.5	Top	0.005654	0.004001	0.004587	0.004621	0.004355
6TH	27	Top	0.005407	0.003733	0.004289	0.004321	0.004069
5TH	23.5	Top	0.005054	0.003405	0.003913	0.003943	0.00371
4TH	20	Top	0.004729	0.003073	0.003563	0.003589	0.003394
3RD	16.5	Top	0.004281	0.002675	0.003129	0.003152	0.003003
2ND	13	Top	0.003684	0.002212	0.002618	0.002637	0.002545
1ST	9.5	Top	0.002917	0.001683	0.002026	0.00204	0.00202
PLINTH	6	Top	0.001354	0.000749	0.00092	0.000926	0.00094
Base	0	Top	0	0	0	0	0

A plot for Storey Drifts of Structures in Hard Soil in Y - Direction with load combination (DL+LL+EQYP) has been shown here

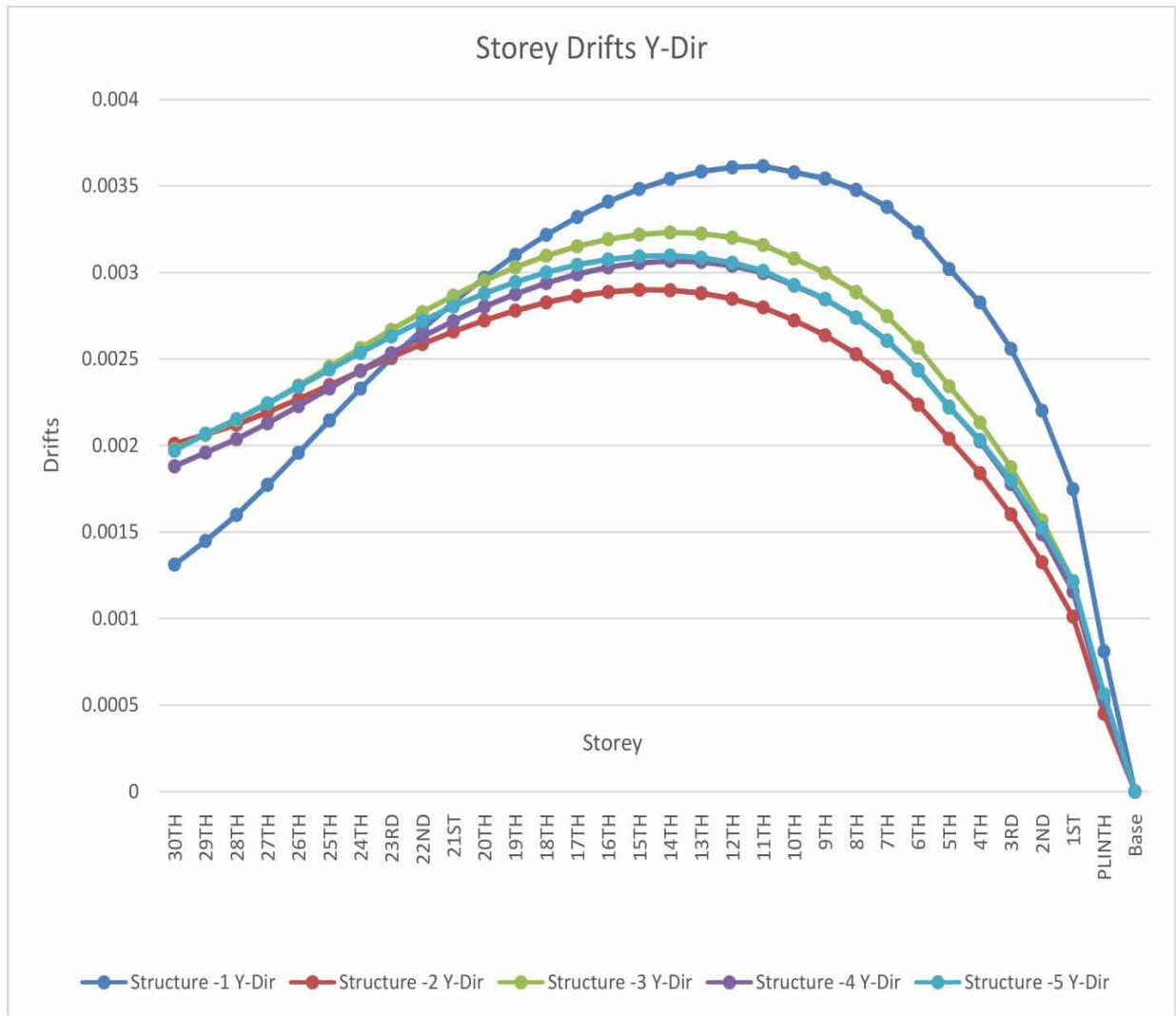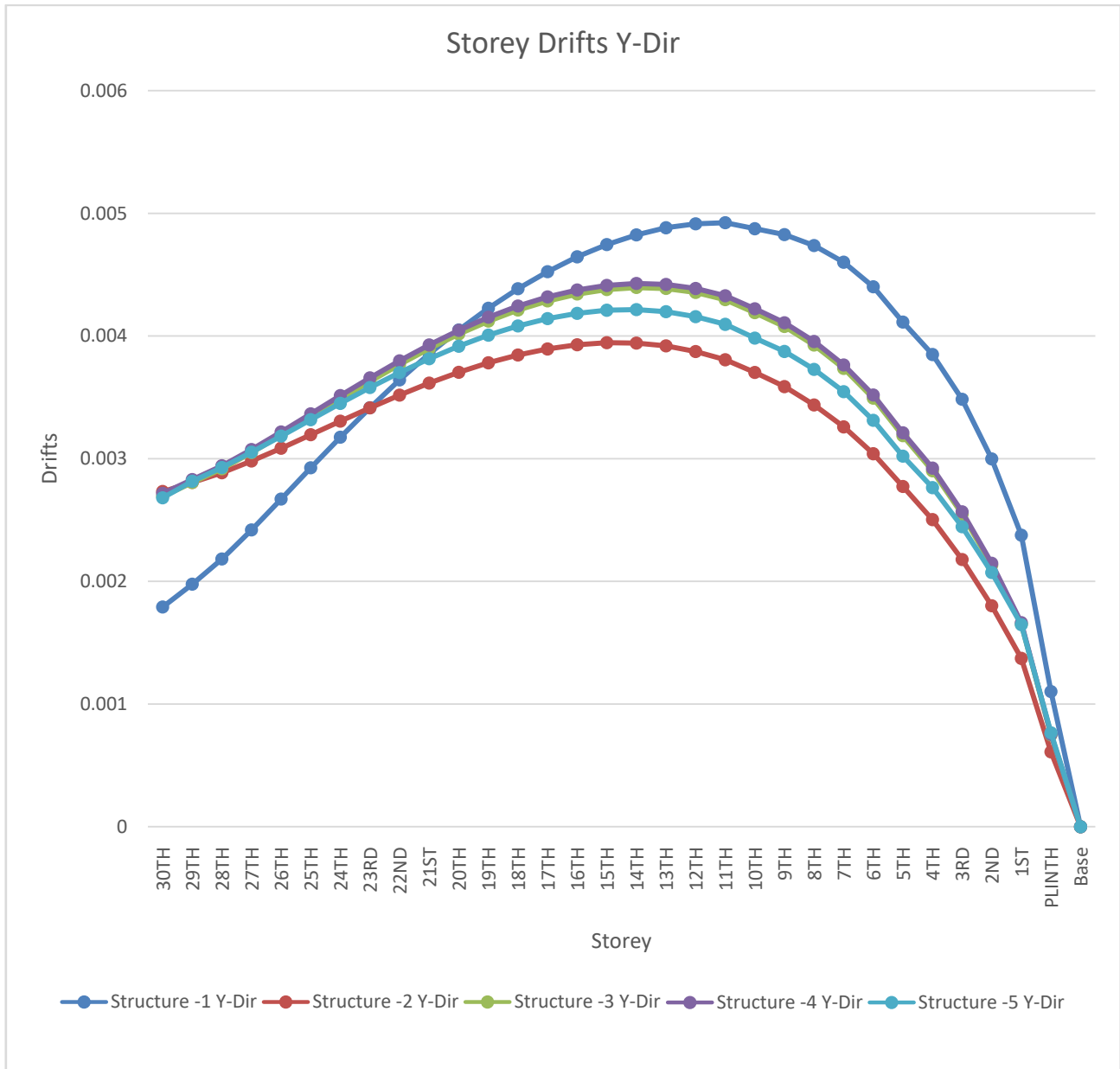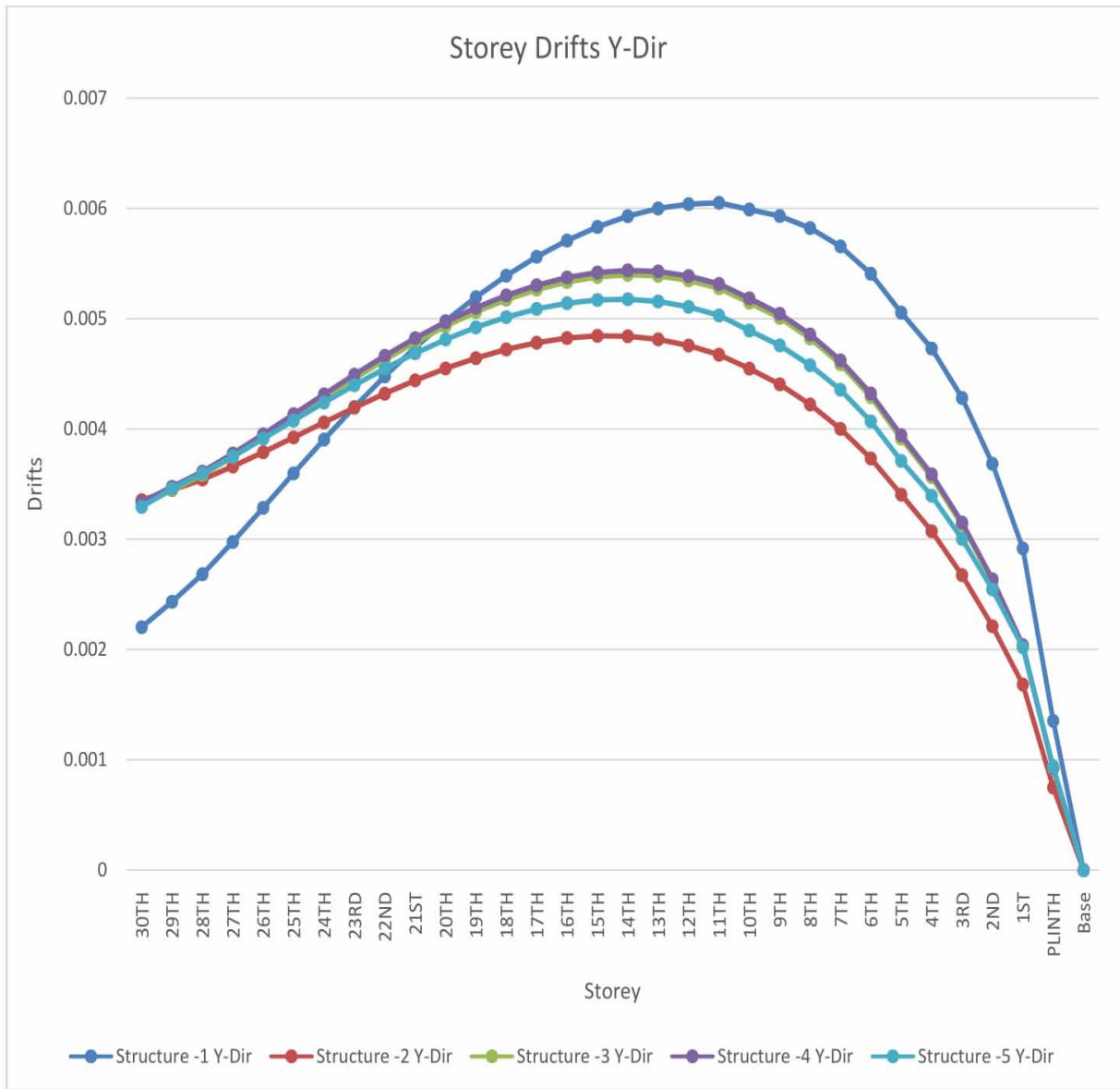*Graph 6:* Storey Drifts of Structures in Hard Soil in Y - Direction

V. Discussion on Results

When a structure is subjected to earthquake, it responds by vibrating. An example force can be resolved into three mutually perpendicular directions-two horizontal directions (X and Y directions) and the vertical direction (Z). This motion causes the structure to vibrate or shake in all three directions; the predominant direction of shaking is horizontal. All the structures are primarily designed for gravity loads-force equal to mass time's gravity in the vertical direction. Because of the inherent factor used in the design specifications, most structures tend to be adequately protected against vertical shaking. Vertical acceleration should also be considered in structures with large spans those in which stability for design, or for overall stability analysis of structures. The basic intent of design theory for earthquake resistant structures is that buildings should be able to resist minor earthquakes without damage, resist moderate earthquakes without structural damage but with some non-structural damage. To avoid collapse during a major earthquake, Members must be ductile enough to absorb and dissipate energy by post elastic deformation. Redundancy in the structural system permits redistribution of internal forces in the event of the failure of key elements. When the primary element or system yields or fails, the lateral force can be redistributed to a secondary system to prevent progressive failure.

The structural prototype is prepared and lots of data is been collected from the prototype. All the aspects such as safety of structure in shear, moment and in story drift have been collected. So now to check whether to know whether the structure is safe with established shear walls and all construction of core wall in the center we need to compare the graphical values of structure with the shear wall and a simple rigid frame structure.

The structures are supported on soil, most of the designers do not consider the soil structure interaction and its subsequent effect on structures during an earthquake. When a structure is subjected to an earthquake excitation, it interacts with the foundation and the soil, and thus changes the motion of the ground. This means that the movement of the whole ground-structure system is influenced by the type of soil as well as by the type of structure. Understanding of soil structure interaction will enable the designer to design structures that will behave better during an earthquake.

a) Story Drift

The tallness of a structure is relative and cannot be defined in absolute terms either in relation to height or the number of stories. The council of Tall Buildings and Urban Habitat considers building having 9 or more stories as high-rise structures. But, from a structural engineer's point of view the tall structure or multi-storied building can be defined as one that, by virtue of its height, is affected by lateral forces due to wind or earthquake or both to an extent. Lateral loads can develop high stresses, produce sway movement or cause vibration. Therefore, it is very important for the structure to have sufficient strength against vertical loads together with adequate stiffness to resist lateral forces. So lateral forces due to wind or seismic loading must be considered for tall building design along with gravity forces vertical loads. Tall and slender buildings are strongly wind sensitive and wind forces are applied to the exposed surfaces of the building, whereas seismic forces are inertial (body forces), which result from the distortion of the ground and the inertial resistance of the building. These forces cause horizontal deflection is the predicted movement of a structure under lateral loads and story drift is defined as the difference in lateral deflection between two adjacent stories. Lateral deflection and drift have three effects on a structure; the movement can affect the structural elements (such as beams and columns); the movements can affect non-structural elements (such as the windows and cladding); and the movements can affect adjacent structures. Without proper consideration during the design process, large deflections and drifts can have adverse effects on structural elements, nonstructural elements, and adjacent structures.

When the initial sizes of the frame members have been selected, an approximate check on the horizontal drift of the structures can be made. The drift in the non-slender rigid frame is mainly caused by racking. This racking may be considered as comprising two components: the first is due to rotation of the joints, as allowed by the double bending of the girders, while the second is caused by double bending of the columns. If the rigid frame is slender, a contribution to drift caused by the overall bending of the frame, resulting from axial deformations of the columns, may be significant. If the frame has height width ratio less than 4:1, the contribution of overall bending to the total drift at the top of the structure is usually less than 10% of that due to racking. The following method of calculation for drift allows the separate determination of the components attributable to beam bending, and overall cantilever action. Drift problem as the horizontal displacement of all tall buildings is one of the most serious issues in tall building design, relating to the dynamic characteristics of the building during earthquakes and strong winds. Drift shall be caused by the accumulated deformations of each member, such as a beam, column and shear wall. In this study analysis is done with changing structural parameters to observe the effect on the drift (lateral deflection) of the tall building due to both wind and earthquake loading. There are three major types of

structures were identified in this study, such as rigid frame, coupled shear wall and wall frame structures.

IS 1893 Part 1 Codal Provoisions for Storey Drift Limitations

The storey drift in any storey due to the minimum specified design lateral force, with partial load factor of 1.0, shall not exceed 0.004 times the storey height For the purposes of displacement requirements only, it is permissible to use seismic force obtained from the computed fundamental period (T) of the building without the lower bound limit on design seismic force specified in dynamic analysis.

The result obtained from the analysis models will be discussed and compared as follows:

b) *It is observed that*

❖ The maximum storey drift in X-direction occurred at storey 13 [th] for structure1 in hard ,medium and soft soil.

❖ The maximum storey drift in X-direction occurred at storey 12 [th] for structure 2 in hard ,medium and soft soil.

❖ The maximum storey drift in X-direction occurred at storey 11 [th] for structure 3 in hard ,medium and soft soil.

❖ The maximum storey drift in X-direction occurred at storey 11 [th] for structure 4 in hard ,medium and soft soil.

❖ The maximum storey drift in X-direction occurred at storey 14 [th] for structure 5 in hard ,medium and soft soil.

Table 8: Comparation Percentage of Story Drifts in Soft soil of Structures 2,3,4,5 with Structure -1

Story Drifts		Structure -2	Structure -3	Structure -4	Structure -5
Load Case/Combo	Direction	%	%	%	%
DLLLEQXP	X	1%	14%	8%	7%
DLLLEQYP	Y	-20%	-8%	-13%	-11%

Table 9: Comparation Percentage of Story Drifts in medium soil of Structures 2,3,4,5 with Structure -1

Story Drifts		Structure -2	Structure -3	Structure -4	Structure -5
Load Case/Combo	Direction	%	%	%	%
DLLLEQXP	X	1%	14%	13%	7%
DLLLEQYP	Y	-20%	-8%	-7%	-11%

Table 10: Comparation Percentage of Story Drifts in hard soil of Structures 2,3,4,5 with Structure -1

Story Drifts		Structure -2	Structure -3	Structure -4	Structure -5
Load Case/Combo	Direction	%	%	%	%
DLLLEQXP	X	1%	14%	13%	7%
DLLLEQYP	Y	-20%	-8%	-7%	-11%

Table 11: Comparation Percentage of Drifts of medium soil and hard soil with soft soil for Structure -1

Structure -1		SOIL TYPE II	SOIL TYPE III
Load Case/Combo	Direction	%	%
DLLLEQXP	X	26%	39%
DLLLEQYP	Y	26%	39%

Table 12: Comparation Percentage of Drifts of medium soil and hard soil with soft soil for Structure -2

Structure -2		SOIL TYPE II	SOIL TYPE III
Load Case/Combo	Direction	%	%
DLLLEQXP	X	26%	39%
DLLLEQYP	Y	26%	39%

Table 13: Comparation Percentage of Drifts of medium soil and hard soil with soft soil for Structure -3

Structure -3		SOIL TYPE II	SOIL TYPE III
Load Case/Combo	Direction	%	%
DLLLEQXP	X	26%	39%
DLLLEQYP	Y	26%	39%

Table 14: Comparation Percentage of Drifts of medium soil and hard soil with soft soil for Structure -4

Structure -4		SOIL TYPE II	SOIL TYPE III
Load Case/Combo	Direction	%	%
DLLLEQXP	X	30%	42%
DLLLEQYP	Y	30%	42%

Table 15: Comparation Percentage of Drifts of medium soil and hard soil with soft soil for Structure -5

Structure -5		SOIL TYPE II	SOIL TYPE III
Load Case/Combo	Direction	%	%
DLLLEQXP	X	25%	39%
DLLLEQYP	Y	26%	39%

VI. CONCLUSIONS

In this paper, reinforced concrete shear wall buildings were analyzed with the procedures laid out in IS codes. Seismic performance of building model is evaluated. In this study, regular shaped structures have been considered. Estimation of drift was carried out for Dual frame system with shear wall structure. This study indicates that the drift on high rise structures has to be considered as it has a notable magnitude. So every tall structure should include the drift due to earthquake load.

From the above results and discussions, following conclusions can be drawn:

1. Building with box shape Shear Walls provided at the center core showed better performance in term of maximum storey drifts.

2. From result observed that drift is increased as height of building increased and reduced at top floor.

3. From the comparison of story drift values it can be observed that maximum reduction in drift values is obtained when shear walls are provided at center of the building.

4. As per code, the actual drift is less than permissible drift. The parallel arrangement of shear wall in the center core and outer periphery is giving very good result in controlling drift in both the direction. The better performance for all the structures with soft soil because it has low storey drift.

5. Storey drifts are found within the limit ,As per Indian standard, Criteria for earthquake resistant design of structures, IS 1893 (Part 1) : 2002, the story drift in any story due to service load shall not exceed 0.004 times the story height.

6. The moment resisting frame with shear walls are very good in lateral force such as earthquake and wind force. The shear walls provide lateral load distribution by transferring the wind and earthquake

loads to the foundation. And also impact on the lateral stiffness of the system and also carries gravity loads.

7. For severe lateral loads caused by wind load and or earthquake load, the reinforced shear wall is obvious. Because, it produces less deflection and less bending moment in connecting beams under lateral loads than all others structural system.

8. Based on the analysis and discussion, shear wall are very much suitable for resisting earthquake induced lateral forces in multistoried structural systems when compared to multistoried structural systems whit out shear walls. They can be made to behave in a ductile manner by adopting proper detailing techniques.

9. ETABS is the advanced software which is used for analysing any kind of building structures. By its fast and accuracy it can easily analyses buildings up to 40 floors.

10. ETABS can analyse any building structure with pre-determined load conditions and load combinations for shear walls regarding IS codes.

11. So, for designing of building shear wall structure if we use ETABS software then it analyse the building easily and give the fast results with accurate data.

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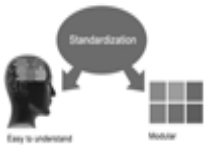
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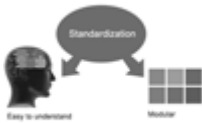
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17. Never copy others' work: Never copy others' work and give it your name because if the evaluator has seen it anywhere, you will be in trouble. Take proper rest and food: No matter how many hours you spend on your research activity, if you are not taking care of your health, then all your efforts will have been in vain. For quality research, take proper rest and food.

18. Go to seminars: Attend seminars if the topic is relevant to your research area. Utilize all your resources.

19. Refresh your mind after intervals: Try to give your mind a rest by listening to soft music or sleeping in intervals. This will also improve your memory. Acquire colleagues: Always try to acquire colleagues. No matter how sharp you are, if you acquire colleagues, they can give you ideas which will be helpful to your research.

20. Think technically: Always think technically. If anything happens, search for its reasons, benefits, and demerits. Think and then print: When you go to print your paper, check that tables are not split, headings are not detached from their descriptions, and page sequence is maintained.

21. Adding unnecessary information: Do not add unnecessary information like "I have used MS Excel to draw graphs." Irrelevant and inappropriate material is superfluous. Foreign terminology and phrases are not apropos. One should never take a broad view. Analogy is like feathers on a snake. Use words properly, regardless of how others use them. Remove quotations. Puns are for kids, not grunt readers. Never oversimplify: When adding material to your research paper, never go for oversimplification; this will definitely irritate the evaluator. Be specific. Never use rhythmic redundancies. Contractions shouldn't be used in a research paper. Comparisons are as terrible as clichés. Give up ampersands, abbreviations, and so on. Remove commas that are not necessary. Parenthetical words should be between brackets or commas. Understatement is always the best way to put forward earth-shaking thoughts. Give a detailed literary review.

22. Report concluded results: Use concluded results. From raw data, filter the results, and then conclude your studies based on measurements and observations taken. An appropriate number of decimal places should be used. Parenthetical remarks are prohibited here. Proofread carefully at the final stage. At the end, give an outline to your arguments. Spot perspectives of further study of the subject. Justify your conclusion at the bottom sufficiently, which will probably include examples.

23. Upon conclusion: Once you have concluded your research, the next most important step is to present your findings. Presentation is extremely important as it is the definite medium though which your research is going to be in print for the rest of the crowd. Care should be taken to categorize your thoughts well and present them in a logical and neat manner. A good quality research paper format is essential because it serves to highlight your research paper and bring to light all necessary aspects of your research.

INFORMAL GUIDELINES OF RESEARCH PAPER WRITING

Key points to remember:

- Submit all work in its final form.
- Write your paper in the form which is presented in the guidelines using the template.
- Please note the criteria peer reviewers will use for grading the final paper.

Final points:

One purpose of organizing a research paper is to let people interpret your efforts selectively. The journal requires the following sections, submitted in the order listed, with each section starting on a new page:

The introduction: This will be compiled from reference matter and reflect the design processes or outline of basis that directed you to make a study. As you carry out the process of study, the method and process section will be constructed like that. The results segment will show related statistics in nearly sequential order and direct reviewers to similar intellectual paths throughout the data that you gathered to carry out your study.

The discussion section:

This will provide understanding of the data and projections as to the implications of the results. The use of good quality references throughout the paper will give the effort trustworthiness by representing an alertness to prior workings.

Writing a research paper is not an easy job, no matter how trouble-free the actual research or concept. Practice, excellent preparation, and controlled record-keeping are the only means to make straightforward progression.

General style:

Specific editorial column necessities for compliance of a manuscript will always take over from directions in these general guidelines.

To make a paper clear: Adhere to recommended page limits.

Mistakes to avoid:

- Insertion of a title at the foot of a page with subsequent text on the next page.
- Separating a table, chart, or figure—confine each to a single page.
- Submitting a manuscript with pages out of sequence.
- In every section of your document, use standard writing style, including articles ("a" and "the").
- Keep paying attention to the topic of the paper.

- Use paragraphs to split each significant point (excluding the abstract).
- Align the primary line of each section.
- Present your points in sound order.
- Use present tense to report well-accepted matters.
- Use past tense to describe specific results.
- Do not use familiar wording; don't address the reviewer directly. Don't use slang or superlatives.
- Avoid use of extra pictures—include only those figures essential to presenting results.

Title page:

Choose a revealing title. It should be short and include the name(s) and address(es) of all authors. It should not have acronyms or abbreviations or exceed two printed lines.

Abstract: This summary should be two hundred words or less. It should clearly and briefly explain the key findings reported in the manuscript and must have precise statistics. It should not have acronyms or abbreviations. It should be logical in itself. Do not cite references at this point.

An abstract is a brief, distinct paragraph summary of finished work or work in development. In a minute or less, a reviewer can be taught the foundation behind the study, common approaches to the problem, relevant results, and significant conclusions or new questions.

Write your summary when your paper is completed because how can you write the summary of anything which is not yet written? Wealth of terminology is very essential in abstract. Use comprehensive sentences, and do not sacrifice readability for brevity; you can maintain it succinctly by phrasing sentences so that they provide more than a lone rationale. The author can at this moment go straight to shortening the outcome. Sum up the study with the subsequent elements in any summary. Try to limit the initial two items to no more than one line each.

Reason for writing the article—theory, overall issue, purpose.

- Fundamental goal.
- To-the-point depiction of the research.
- Consequences, including definite statistics—if the consequences are quantitative in nature, account for this; results of any numerical analysis should be reported. Significant conclusions or questions that emerge from the research.

Approach:

- Single section and succinct.
- An outline of the job done is always written in past tense.
- Concentrate on shortening results—limit background information to a verdict or two.
- Exact spelling, clarity of sentences and phrases, and appropriate reporting of quantities (proper units, important statistics) are just as significant in an abstract as they are anywhere else.

Introduction:

The introduction should "introduce" the manuscript. The reviewer should be presented with sufficient background information to be capable of comprehending and calculating the purpose of your study without having to refer to other works. The basis for the study should be offered. Give the most important references, but avoid making a comprehensive appraisal of the topic. Describe the problem visibly. If the problem is not acknowledged in a logical, reasonable way, the reviewer will give no attention to your results. Speak in common terms about techniques used to explain the problem, if needed, but do not present any particulars about the protocols here.

The following approach can create a valuable beginning:

- Explain the value (significance) of the study.
- Defend the model—why did you employ this particular system or method? What is its compensation? Remark upon its appropriateness from an abstract point of view as well as pointing out sensible reasons for using it.
- Present a justification. State your particular theory(-ies) or aim(s), and describe the logic that led you to choose them.
- Briefly explain the study's tentative purpose and how it meets the declared objectives.

Approach:

Use past tense except for when referring to recognized facts. After all, the manuscript will be submitted after the entire job is done. Sort out your thoughts; manufacture one key point for every section. If you make the four points listed above, you will need at least four paragraphs. Present surrounding information only when it is necessary to support a situation. The reviewer does not desire to read everything you know about a topic. Shape the theory specifically—do not take a broad view.

As always, give awareness to spelling, simplicity, and correctness of sentences and phrases.

Procedures (methods and materials):

This part is supposed to be the easiest to carve if you have good skills. A soundly written procedures segment allows a capable scientist to replicate your results. Present precise information about your supplies. The suppliers and clarity of reagents can be helpful bits of information. Present methods in sequential order, but linked methodologies can be grouped as a segment. Be concise when relating the protocols. Attempt to give the least amount of information that would permit another capable scientist to replicate your outcome, but be cautious that vital information is integrated. The use of subheadings is suggested and ought to be synchronized with the results section.

When a technique is used that has been well-described in another section, mention the specific item describing the way, but draw the basic principle while stating the situation. The purpose is to show all particular resources and broad procedures so that another person may use some or all of the methods in one more study or referee the scientific value of your work. It is not to be a step-by-step report of the whole thing you did, nor is a methods section a set of orders.

Materials:

Materials may be reported in part of a section or else they may be recognized along with your measures.

Methods:

o Report the method and not the particulars of each process that engaged the same methodology.
o Describe the method entirely.
o To be succinct, present methods under headings dedicated to specific dealings or groups of measures.
o Simplify—detail how procedures were completed, not how they were performed on a particular day.
o If well-known procedures were used, account for the procedure by name, possibly with a reference, and that's all.

Approach:

It is embarrassing to use vigorous voice when documenting methods without using first person, which would focus the reviewer's interest on the researcher rather than the job. As a result, when writing up the methods, most authors use third person passive voice.

Use standard style in this and every other part of the paper—avoid familiar lists, and use full sentences.

What to keep away from:

o Resources and methods are not a set of information.
o Skip all descriptive information and surroundings—save it for the argument.
o Leave out information that is immaterial to a third party.

Results:

The principle of a results segment is to present and demonstrate your conclusion. Create this part as entirely objective details of the outcome, and save all understanding for the discussion.

The page length of this segment is set by the sum and types of data to be reported. Use statistics and tables, if suitable, to present consequences most efficiently.

You must clearly differentiate material which would usually be incorporated in a study editorial from any unprocessed data or additional appendix matter that would not be available. In fact, such matters should not be submitted at all except if requested by the instructor.

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XVI

Content:

- Sum up your conclusions in text and demonstrate them, if suitable, with figures and tables.
- In the manuscript, explain each of your consequences, and point the reader to remarks that are most appropriate.
- Present a background, such as by describing the question that was addressed by creation of an exacting study.
- Explain results of control experiments and give remarks that are not accessible in a prescribed figure or table, if appropriate.
- Examine your data, then prepare the analyzed (transformed) data in the form of a figure (graph), table, or manuscript.

What to stay away from:

- Do not discuss or infer your outcome, report surrounding information, or try to explain anything.
- Do not include raw data or intermediate calculations in a research manuscript.
- Do not present similar data more than once.
- A manuscript should complement any figures or tables, not duplicate information.
- Never confuse figures with tables—there is a difference.

Approach:

As always, use past tense when you submit your results, and put the whole thing in a reasonable order.

Put figures and tables, appropriately numbered, in order at the end of the report.

If you desire, you may place your figures and tables properly within the text of your results section.

Figures and tables:

If you put figures and tables at the end of some details, make certain that they are visibly distinguished from any attached appendix materials, such as raw facts. Whatever the position, each table must be titled, numbered one after the other, and include a heading. All figures and tables must be divided from the text.

Discussion:

The discussion is expected to be the trickiest segment to write. A lot of papers submitted to the journal are discarded based on problems with the discussion. There is no rule for how long an argument should be.

Position your understanding of the outcome visibly to lead the reviewer through your conclusions, and then finish the paper with a summing up of the implications of the study. The purpose here is to offer an understanding of your results and support all of your conclusions, using facts from your research and generally accepted information, if suitable. The implication of results should be fully described.

Infer your data in the conversation in suitable depth. This means that when you clarify an observable fact, you must explain mechanisms that may account for the observation. If your results vary from your prospect, make clear why that may have happened. If your results agree, then explain the theory that the proof supported. It is never suitable to just state that the data approved the prospect, and let it drop at that. Make a decision as to whether each premise is supported or discarded or if you cannot make a conclusion with assurance. Do not just dismiss a study or part of a study as "uncertain."

Research papers are not acknowledged if the work is imperfect. Draw what conclusions you can based upon the results that you have, and take care of the study as a finished work.

- You may propose future guidelines, such as how an experiment might be personalized to accomplish a new idea.
- Give details of all of your remarks as much as possible, focusing on mechanisms.
- Make a decision as to whether the tentative design sufficiently addressed the theory and whether or not it was correctly restricted. Try to present substitute explanations if they are sensible alternatives.
- One piece of research will not counter an overall question, so maintain the large picture in mind. Where do you go next? The best studies unlock new avenues of study. What questions remain?
- Recommendations for detailed papers will offer supplementary suggestions.

Approach:

When you refer to information, differentiate data generated by your own studies from other available information. Present work done by specific persons (including you) in past tense.

Describe generally acknowledged facts and main beliefs in present tense.

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Topics	Grades		
	A-B	**C-D**	**E-F**
Abstract	Clear and concise with appropriate content, Correct format. 200 words or below	Unclear summary and no specific data, Incorrect form Above 200 words	No specific data with ambiguous information Above 250 words
Introduction	Containing all background details with clear goal and appropriate details, flow specification, no grammar and spelling mistake, well organized sentence and paragraph, reference cited	Unclear and confusing data, appropriate format, grammar and spelling errors with unorganized matter	Out of place depth and content, hazy format
Methods and Procedures	Clear and to the point with well arranged paragraph, precision and accuracy of facts and figures, well organized subheads	Difficult to comprehend with embarrassed text, too much explanation but completed	Incorrect and unorganized structure with hazy meaning
Result	Well organized, Clear and specific, Correct units with precision, correct data, well structuring of paragraph, no grammar and spelling mistake	Complete and embarrassed text, difficult to comprehend	Irregular format with wrong facts and figures
Discussion	Well organized, meaningful specification, sound conclusion, logical and concise explanation, highly structured paragraph reference cited	Wordy, unclear conclusion, spurious	Conclusion is not cited, unorganized, difficult to comprehend
References	Complete and correct format, well organized	Beside the point, Incomplete	Wrong format and structuring

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