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NON-LINEAR ANALYSIS OF MULTISTOREY G + 4 BUILDING BY TIME HISTORY METHOD USING NEWMARK'S LINEAR AND AVERAGE ACCELERATION METHOD

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ABSTRACT
This paper presents the non linear dynamic analysis of G + 4 storey building using Newmark's time stepping methods. The ground acceleration data used for this study was EL Centro values. A total of 1559 ground acceleration values are considered for calculation of peak lateral force, shear force, displacement for each floor and these results are compared with static analysis of the same considered building as per IS 1893(PART I) ; 2002. Results showed that the there is a need for improvement in analysis of ground soft storey structures where the buildings week and tend to critical effect due to earthquake effect.

KEYWORDS: Dynamic analysis, static analysis, Newmark's time stepping method

INTRODUCTION
Earthquakes result in ground motion both horizontal and vertical which can be compared to waves. The motion is generally vibratory and will cause a structure to move rapidly first in one direction and then other.

Earthquakes generate internal forces in a structure due to inertia. Inertia can be described as the tendency of a body at rest to remain at rest and a body in motion remains in motion. The internal forces depend on the direction of ground motion caused by an earthquake and act horizontal and vertical.

The more pronounced earthquake forces are usually horizontal i.e. lateral forces acting back and forth parallel to the ground. Because the ground motion moves back and forth, the effects of inertia cause a building to be distorted and can result in severe damage. The effects of vertical acceleration are normally considered to be offset by the building weight and will only cause damage in unusual situations.

LITERATURE REVIEW
Juan C. Reyes et al [2014] made an assessment of spectrum matching procedure for nonlinear analysis of symmetric and asymmetric plan of building with usage of ASCE/SEI 7-10 scaling procedure with unscaled records. 30 ground motion studies are considered for this analysis of earthquake names are San Fernando (1971), Imperial Valley earthquakes (1979), Irpinia,Italy (1980), superstition hills (1987), Loma prieta (1989), Northridge (1984), and Kobe Japan earthquake (1995), these are the earthquakes ranging from 6.5 magnitude to 6.9 magnitude of NEHRP site class of C and D. The analysis is made both for single story building and multi-story buildings and stated that nonlinear analysis for symmetric and asymmetric plan of buildings provides accurate EDP values with comparison with rigorous bench mark.

R.C.Soares et al., [2002] formulated the reliability of reinforced concrete structures using response surface method. Structural reliability index is made using rackwitz and fiessler algorithm which have a reduced amount of iterations. Parametric numeral analysis for columns and frames are made. This attempt concluded that RSM provides high capability to estimate reliability index of nonlinear reinforced concrete structure. As reducing number of iterations also the method gave accurate results to reliability indexes. This attempt showed that calculations of reliability index for the building elements is very difficult task requires more attention in getting data of buildings and structural elements from material properties. The partial safety factors adopted for calculation of columns and frames for concrete, steel, loads are 1.4, 1.15, and 1.40 respectively. Frame deterministic values of the considered building are identified and tabulated; those are cross section of columns, beams, reinforced position, steel mean strength and steel young modulus.

Nikos D Lagaros [2012] made a neural network prediction scheme for nonlinear seismic response of 3D buildings, of reinforced concrete and steel buildings. A 2, 3, 6 story 3D RC building analysis is made using ANN techniques considering artificial accelerograms, where Tm is taken as 72, 475, 2475 years. In considering of occasional earthquake hazard level, rare earthquake hazard level, maximum considered event earthquake hazard level with 50%, 10%, and 2% of occurrence interval of earthquake respectively.

N.Nacahira et al [1990] using newmark numerical method, designed an alternative approach can be used for manual calculations for designing. The simplicity of method is demonstrated by taking warren truss and gerberturss and results are compared with FEM. Presented a practical application for calculation of fundamental frequencies for plane trusses. Elastic weights of plane trusses are obtained by principle of virtual displacements but weights are not always presented. Procedure is given as follows:

Step1: trail vertical deflections are assumed and inertia forces at each joint are calculated.

Step 2: Weights are calculated from the prescribed tabular form.

Step 3: member forces are calculated from the inertial forces.

Step 4: Beam bending moments are calculated using newmark's equations.

Step 5: if correct deflections are assumed then the iterations will be closed form, if not trail deflections are assumed which coincides with new deflections.

Step 6: Using the converged results, natural frequencies are obtained.

And calculated the vibration of simple warren truss with four panels with four panels with EA constant. Vibrations of Gerber's truss with three spans having vertical Members and found that good accuracy in comparison with FEM are obtained by the Formulated procedure.

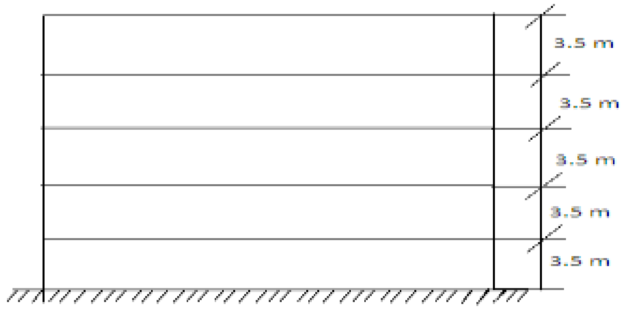
Jacques Ingles et al [2006] studied the effects of vertical component of ground shaking on earthquake induced land slide displacements using newmarks analysis with consideration of two important limitations (1) considering vertical acceleration only. (2) Inclination of slope only refers to ratio of vertical to horizontal acceleration. Noticeable difference was observed between displacements and ground accelerations resulted from newmarks method. The important concepts observed are: when D is greater than critical displacement of slope at same time, newmarks critical acceleration and seismic horizontal critical acceleration, taking into account the vertical component of ground shaking are even greater than for planar slip surfaces.

Ilknur Bozbey, Ozgun Gundogdu [2011] proposed a methodology to select seismic coefficients based on upper bound Newmarks displacement using earthquake records in turkey. A total of 49 strong motion records taken during 37 earthquakes in turkey and are used for calculation of slope displacements for different acceleration ratios. A special code named quake analyzer is used for this study to calculate Newmark displacements. The range of earthquakes considered for the analysis procedure are magnitude of earthquakes ranging from 6M to 7M.

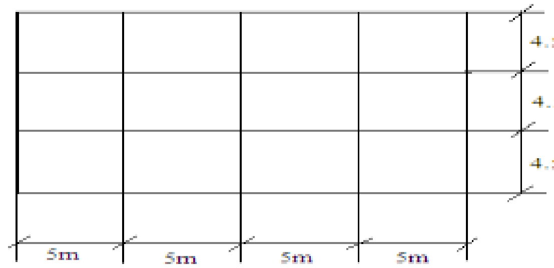
Steen Krenk [2006] demonstrated the stability limit in newmark algorithm reaches when the stiffness term in newmark based time algorithm vanishes energy fluctuations closes to a stability limit. For analysis of newmark based time integration algorithm, used set of different relations of displacement, acceleration and velocity vectors. Showed a clear step by step procedure for energy balance of newmark algorithm, without structural damping, stability response and modal response, energy balance with structures damping and energy balance for generalized newmark algorithms. Proved that newmark time integration algorithm satisfies energy balance equation.

S.M.Wilkinson, R.A.Hiley [2006] developed a nonlinear response history model for seismic analysis of high rise framed buildings. This model suits the buildings with m(n+2) where m is the number of stories and n is the number of bays with rank of stiffness matrix. This model allows to have multiple redundancies and connections with moment rotation relationship. The model was developed with axial compressive forces, elastic stiffness matrix for beams neglecting column axial degree of freedom, with corresponding vector of displacements. Deformation of beam column connections and plastic end rotation remains constant under elastic loading and unloading. Damping coefficient was set to 5% for all modes for high rise buildings.

PLAN AND ELEVATION



Elevation of the considered building



plan of the considered building

DESIGN DATA

Thickness of slab = 0.13 m
Load due to roof finish = 3 kN/m^2
Load due to floor finish = 2 kN/m^2
Thickness of outer walls = 250mm
Thickness of inner walls = 150 mm
Imposed load = 3.5 kN/m^2
Type of foundation = isolated footing
Length in x-direction = 20 m
Length in z-direction = 13.5 m
No of bays in x-direction = 4
No of bays in z-direction = 3
Total floor area = 270 m^2
Floor to floor height = 3.5 m
No of floors without stilt = 4
Total height of the building = 17.5 m
Unit wt of masonry = 20
Unit wt of concrete = 25
The seismic weight of the building is calculated as per
IS 1893 PART I : 2002.
Total seismic weight of the building calculated is 14919kN
Fundamental time period is $T_a = 0.09*17.5/\sqrt{20} = 0.352$ sec
Average response acceleration coefficient is $S_a/g = 2.5$
Zone factor considered for the building in calculation for static procedure is Z = 0.36
Importance factor considered is I = 1
Response reduction factor is R = 5
Calculated lateral force is shown in table no: 1.

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Table 1DESIGN LATERAL FORCE FOR ZONE V

Storey	Wi kN	Hi (m)	Wi x Hi^2	(Wi x Hi^2)/(Σ Wi x Hi^2)	Lateral force KN
4	2282.25	17.5	698939.1	0.363	488.64
3	3349.5	14	656502	0.341	458.97
2	3349.5	10.5	369282.4	0.192	258.17
1	3349.5	7.0	161425.5	0.085	114.74
0	2588.25	3.5	31706.6	0.016	22.166

Non linear analysis of the considered G + 4 building is done using Mat lab program.

From the matlab program, output graphs of displacement, lateral force, and shear force are obtained for 1559 EL Centro values with respect to increment of time 0.02 sec.

COMPARISION OF RESULTS

The following tabular form shows the results of the average acceleration method, linear acceleration method based on the nonlinear analysis equations of NEWMARK's method equations which are adopted from DYNAMICS OF STRUCTURES by A.K.Chopra text book.the equivalent static method is also done for the same considered G + 4 building as per IS : 1893:2002 (PART I) with zone value of V, IV, III, II for response reduction factor R = 5 and time period of 2.5sec, and the design lateral force at each floor results are compared with average acceleration method, linear acceleration method results.

Fig 1. GRAPH SHOWS THE PATTERN OF CHANGE OF BASE SHEAR WITH HEIGHT FOR ZONE V

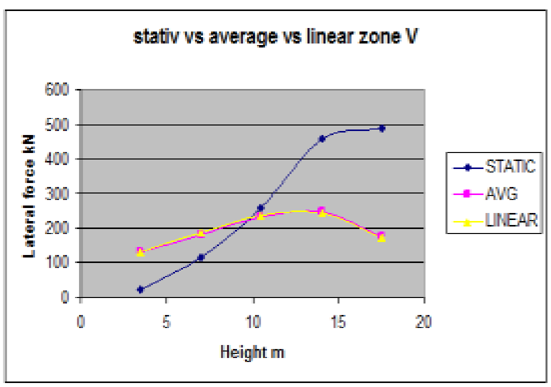


Fig 2. GRAPH SHOWS THE PATTERN OF CHANGE OF BASE SHEAR WITH HEIGHT FOR ZONE IV

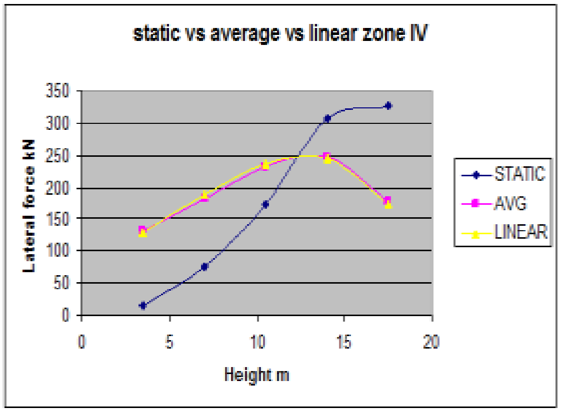


Fig 3. Graph for change of base shear with height for zone III.

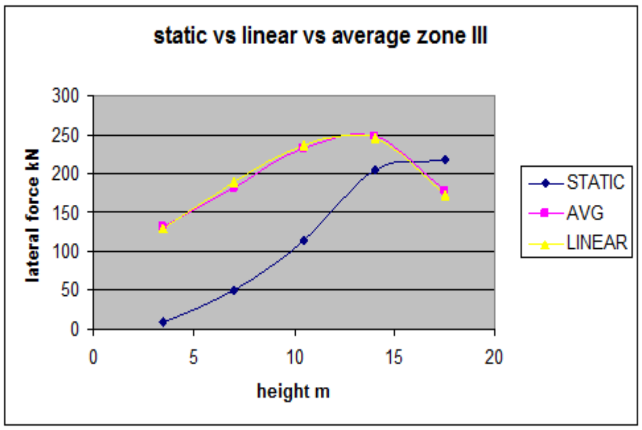
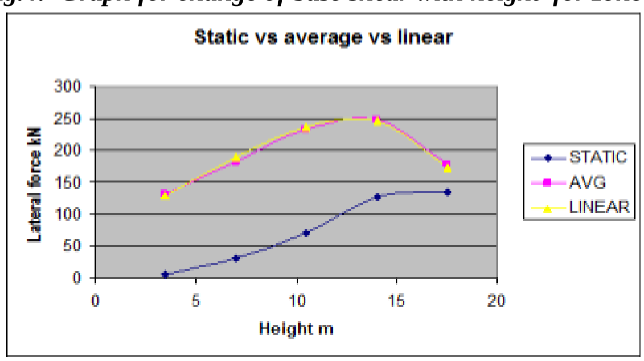


Fig.4: Graph for change of base shear with height for zone II



CONCLUSIONS

Here for the considered project we done the analysis procedure as specified in the previous steps and given the comparative results for every storey and every method which includes graphical representation also so in the process of this analysis we observed a point which will give a conclusion to this project as the lateral force obtained in zone IV is 7.8% less when compared with average acceleration method and linear acceleration method respectively.

In both the case of zone V and zone IV the lateral force obtained at silt level and first floor there is a much high difference in lateral force when compared with average acceleration method and linear acceleration method. So, by showing the above comparative result for different zone in different methods here we concluded that there is a need for strengthening of ground levels in Indian static condition as when compared with the ground acceleration generated lateral force.

When compared to zone V and zone IV graphs the base shear was more at levels of 4^{th} floor to 5^{th} floor when compared with linear acceleration method to average acceleration method. But at the same levels in zone III and zone II the lateral force is more in linear and average acceleration methods.

From the observations of all the graphs a major value of lateral force, shear, and displacement was observed at the first 10 seconds of iteration time period in a value of 31 seconds considered for the project.

The time period 31 seconds was the time accumulated with the iteration of the time interval considered for the project of 0.02 seconds. The loop of time interval starts with zero seconds and ends with 31 seconds, similarly the obtained lateral force, displacement, shear force at the starting of the loop are zero and at the end of the loop of the ground acceleration values is also zero.

The definition of the project work is to identify the base shear, displacement, acceleration values for the buildings by non linear analysis which are made with one component matched of ground motions, is independent of building behavior.

The usage of non linear analysis with time history method and with using ground acceleration of previous data records from the data can be compared with the zone intensities of the earthquake zones of India and can be used for safe vulnerability of earthquake analysis. But for analysis at least 7 ground accelerations are to be collected and are to analyzed for the building to get matched spectrum.

But the Nonlinear analysis of the framed structures is more expensive when compared with linear methods and static methods as it consumes more time than required for static methods which results in final construction cost of the project. The analysis requires adequate scientific data of earthquake when used by iteration methods. Static and dynamic shear resistances of the materials considered are same and constant.

When displacements D are greater than the critical displacements of the considered site condition, the calculation is done with the same time interval of 0.02 seconds. As Newmark's analysis predicts slope stability of the site conditions. Hence Newmark's method can be concluded for its impossibility for pale landslide conditions.

There is a great scatter in displacement values for the same ground acceleration values considered with time interval of 0.02 seconds ranging from 0.001 to over 1000cm for the considered G + 4 building.

The distribution of parameters like acceleration, velocity, displacement, base shear, and shear force for the story diaphragms is not sensitive to changes in the parameters considered. The same is true for lateral displacements of the individual frames. Generalized masses in the fundamental modes of the structure and the non-structural elements are equal to their respective total masses.

FUTURE SCOPE

The present work details the deficiency of the general method Equivalent static method regarding identifying the lateral force correctly for each and every floor. The work is based on the consideration of real ground data which showed a much difference of lateral force with equivalent static method of IS : 1893 (PART I:2002), as vise there is a need for calculation of lateral forces and check for stability of buildings for real ground acceleration values, which will increase the understanding capacity of effect of earthquake on the structures, enhances the safety of structures. There is a need to make adequate ground acceleration data of earthquakes occurred in India and similar earthquake acceleration data of the countries, available by the government of India for all the structural engineers and researchers of the country to make safe India and encourage the strength of knowledge in seismic engineering and technology.

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MAT LAB PROGRAM

```
% PROGRAM FOR NEWMARKS NON LINEAR PROCEDURE: Average acceleration method:
clear all
clc
format long
nobx=4;%input('Enter no of bays along X-axis: ');
lx=5;%input('specify length of each bay along X-axis(in M): ');
noby=5;%input('Enter no of bays along Y-axis: ');
ly=3.5;%input('specify length of each bay along Y-axis(in M): ');
nobz=3;%input('Enter no of bays along Z-axis: ');
lz=4.5;%input('specify length of each bay along Z-axis(in M): ');
b=0.3;%input('specify width of beam: ');
dx=0.4;%input('specify depth of beam: ');
dz=0.4;
c1=0.25;%input('specify width of column: ');
c2=0.45;%input('specify depth of column: ');
ds=0.13;%input('Specify depth of slab: ');
Ec=22360000;%input('specify Youngs modulus of material: ');
Em=13800000;
twx=0.25;twz=0.15;
ll=3.5;
%Node coordinate matrix. . .
NON=(nobx+1)*(noby+1)*(nobz+1);
NC=zeros(NON,4);
n=1;
for k=1:nobz+1
   for j=1:noby+1
      for i=1:nobx+1
         NC(n,1)=n;
         NC(n,2)=(i-1)*lx;
         NC(n,3)=(j-1)*ly;
         NC(n,4)=(k-1)*lz;
         n=n+1;
      end
   end
end
NC'
% Element connectivity matrix. . .
NOE=((nobx+1)*noby+nobx*noby)*(nobz+1)+(nobz*noby)*(nobx+1);
ECM=zeros(NOE,3);
n=1;
for k=1:nobz+1
   for j=1:noby
      for i=1:nobx+1
         ECM(n,1)=n;
         ECM(n,2)=(k-1)*((nobx+1)*(noby+1))+(j-1)*(nobx+1)+i;
         ECM(n,3)=ECM(n,2)+nobx+1;
         n=n+1;
      end
   end
   for j=1:noby
      for i=1:nobx
         ECM(n,1)=n;
         ECM(n,2)=(k-1)*((nobx+1)*(noby+1))+(j-1)*(nobx+1)+i+nobx+1;
```

```
            ECM(n,3)=ECM(n,2)+1;
            n=n+1;
        end
    end
end
n=(nobz+1)*((nobx+1)*noby+nobx*noby)+1;
for k=1:nobz
    for j=1:noby
        for i=1:nobx+1
            ECM(n,1)=n;
            ECM(n,2)=(k-1)*((nobx+1)*(noby+1))+(j-1)*(nobx+1)+i+nobx+1;
            ECM(n,3)=ECM(n,2)+(nobx+1)*(noby+1);
            n=n+1;
        end
    end
end
ECM'
fc=25000;fy=415000;gc=25;gm=20;
Ib=(b*dx^3)/12;
Ic=(c1*c2^3)/12;
theeta=atan(ly/lx);
alphah=(pi/2)*(Ec*Ic*ly/(2*Em*twx*sin(2*theeta)))^0.25;
alphal=pi*(Ec*Ib*lx/(Em*twx*sin(2*theeta)))^0.25;
wd=0.5*sqrt(alphah^2+alphal^2);
ld=sqrt(ly^2+lx^2);
kd=(wd*twx*Em/ld)*(cos(theeta))^2;
Lx=nobx*lx;Lz=nobz*lz;H=noby*ly;
zi=0.05;
if ll<3
    lf=0.25;
else
    lf=0.5;
end

% Calculation of Mass and Stiffness matrices
nos=noby;
M=zeros(nos);
K=zeros(nos);
for i=1:noby
    if i==noby

m(i)=(b*dx*Lx+(nobx+1)*b*dz*Lz)*gc+ds*Lx*Lz*gc+(nobx+1)*c1*c2*(ly/2)*gc+(twx*Lx*(ly/2)+twz*(nobx+1)
*Lz*(ly/2))*gm;
        k(i)=((nobx+1)*12*Ec*Ic)/(ly^3)+nobx*kd;
    else
m(i)=(b*dx*Lx+(nobx+1)*b*dz*Lz)*gc+ds*Lx*Lz*gc+(nobx+1)*c1*c2*ly*gc+(twx*Lx*ly+twz*(nobx+1)*Lz*ly
)*gm+lf*ll*Lz*Lx;
        k(i)=((nobx+1)*12*Ec*Ic)/(ly^3)+nobx*kd;
    end
end
m=m/9.81;
M=[diag(m)];
disp('Mass Matrix:')
M
```

```
n=1;
for i=1:nos
    for j=1:nos
        if i==j+1
            K(i,j)=-k(n);
        end
        if i==j
            if i==nos
                K(i,j)=k(n);
            else
                K(i,j)=k(n)+k(n+1);
            end
        end
        if i==j-1
            K(i,j)=-k(n+1);
        end
    end
    n=n+1;
end
disp('Stiffness Matrix:')
K
[u,e]=eig(K,M);
disp('The Eigen values and Mode shapes are as follows:\n');
u
e
%Natural freequency (rad/s)
f=sqrt(e);
disp('Natural Freequencies of each mode');
f
%Natural Time period (sec)
T1=[];
for i=1:nos
    T1=horzcat(T1,2*pi/f(i,i));
end
disp('Natural Time Periods (s)');
t=diag(T1)
% Calculation of modal participation factor Claus 7.8.4.5 (b)
P=zeros(1,nos);
p1=zeros(1,nos);
p2=zeros(1,nos);
for i=1:nos
    for j=1:nos
        p1(i)=p1(i)+m(j)*u(j,i);
        p2(i)=p2(i)+m(j)*u(j,i)*u(j,i);
    end
    P(i)=p1(i)/p2(i);
end
disp('Modal 1Participation factor')
P
MF=(u'*M*u);
K1=diag(u'*K*u);
M1=diag(u'*M*u);
C=diag(2*zi*MF*f);
```

```
%Design Accelaration Spectrum for desired earthquake. . .
A=load('D:\gm.txt');
dt=0.02;
N=numel(A)*dt;
figure(1)
hold on
T(1)=[0];
n=2;
for i=1:numel(A)-1
   T(n)=i*dt;
   n=n+1;
end
plot(T,A,'b');
title('Design Accelration Spectrum','Color','r');
xlabel('Time (sec)');
ylabel('Accelaration (m/sq-sec)');

%Normalised coordinate or displacement spectrum
x=zeros(nos,numel(A));
xd=zeros(nos,numel(A));
xdd=zeros(nos,numel(A));
Xmax=zeros(nos,1);
for i=1:nos
   figure(i+1)
   hold on
   title(['Displacement Response in Normal coordinates :',int2str(i)],'Color','r');
   m1=M1(i);
   k1=K1(i);
   c=C(i);
   Pt=P(i)*A;
   gaama=1/2;beeta=1/4;
   kcap=k1+gaama*c/(beeta*dt)+m1/(beeta*dt*dt);
   a=m1/(beeta*dt)+gaama*c/beeta;
   b=m1/(2*beeta)+dt*c*(gaama/(2*beeta)-1);
   DP(1)=Pt(2)-Pt(1);
   DPcap(1)=DP(1)+a*xd(i,1)+b*xdd(i,1);
   Dx=DPcap(1)/kcap;
   Dxd=gaama*Dx/(beeta*dt)-gaama*xd(i,1)/beeta-dt*xdd(i,1)*(1-gaama/(2*beeta));
   Dxdd=Dx/(beeta*dt*dt)-xd(i,1)/(beeta*dt)-xdd(i,1)/(2*beeta);
   x(i,2)=x(i,1)+Dx;
   xd(i,2)=xd(i,1)+Dxd;
   xdd(i,2)=xdd(i,1)+Dxdd;
   for j=2:numel(A)-1
      DP(j)=Pt(j+1)-Pt(j);
      DPcap(j)=DP(j)+a*xd(i,j)+b*xdd(i,j);
      Dx=DPcap(j)/kcap;
      Dxd=gaama*Dx/(beeta*dt)-gaama*xd(i,j)/beeta+dt*xdd(i,j)*(1-gaama/(2*beeta));
      Dxdd=Dx/(beeta*dt*dt)-xd(i,j)/(beeta*dt)-xdd(i,j)/(2*beeta);
      x(i,j+1)=x(i,j)+Dx;
      xd(i,j+1)=xd(i,j)+Dxd;
      xdd(i,j+1)=xdd(i,j)+Dxdd;
   end
   plot(T,x(i,:),'r')
   xlabel('Time (sec)');
```

```
   ylabel('X(t) (m)');
   Xmax(i)=max(abs(x(i,:)));
end

%Displacement Response in Physical coordinates
q=zeros(nos,numel(A));
Qmax=zeros(nos,1);
for k=1:nos
   for j=1:nos
      q(k,:)=q(k,:)+u(k,j)*x(j,:);
   end
   figure(k+5)
   hold on
   title(['Displacement Response for storey :',int2str(k)],'Color','r');
   plot(T,q(k,:),'y')
   xlabel('Time (sec)');
   ylabel('Displacement (m)');
   Qmax(k)=max(abs(q(k,:)));
end

%DESIGN LATERAL FORCE ENVOLOP
F=zeros(nos,numel(A));
Fmax=zeros(nos,1);
for k=1:nos
   for j=1:nos
      F(k,:)=F(k,:)+K(k,j)*q(j,:);
   end
   figure(k+9)
   hold on
   title(['Effective EQ Force Response for storey :',int2str(k)],'Color','r');
   plot(T,F(k,:),'g')
   xlabel('Time (sec)');
   ylabel('Effective force (kN)');
   Fmax(k)=max(abs(F(k,:)));
end

%STOREY SHEAR RESPONSE OF EACH STOREY
S=triu(ones(nos));
V=zeros(nos,numel(A));
Vmax=zeros(nos,1);
for k=1:nos
   figure(k+13)
   hold on
   title(['Storey Shear Response for storey :',int2str(k)],'Color','r');
   for j=1:nos
      V(k,:)=V(k,:)+S(k,j)*F(j,:);
   end
   plot(T,V(k,:),'m')
   xlabel('Time (sec)');
   ylabel('Storey Shear (kN)');
   Vmax(k)=max(abs(V(k,:)));
end

%PLOTING OF VARRIATIONS IN EACH STOREY
```

```
i1=1:5;
figure(18)
plot(i1,Xmax(:,1),'r')
figure(19)
plot(i1,Qmax(:,1),'r')
figure(20)
plot(i1,Fmax(:,1),'r')
figure(21)
plot(i1,Vmax(:,1),'r')
xlswrite('D:\disp',x');
xlswrite('D:\force',F');
xlswrite('D:\shear',V');
```

(By changing the ϒ=1/2 and β=1/6 values in above program we will get linear acceleration method result
).