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A NONLINEAR TIME HISTORY ANALYSIS OF TEN STOREY RCC BUILDING

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Abstract - *This research paper describe the result of the time history analysis on the ten storey structure. Earthquake occurred in multistoried building shows that if the structures are not well designed and constructed with and adequate strength it leads to the complete collapse of the structures. To ensure safety against seismic forces of multi-storied building hence, there is need to study of seismic analysis to design earthquake resistance structures.*

TIME HISTORY ANALYSIS provides for linear or nonlinear evaluation of dynamic structural response under loading which may vary according to the specified time function. Dynamic equilibrium equations, given by $K\,u(t) + C\,{}^{d}/_{dt}\,u(t) + M\,d^2/_{dt}\,u(t) = r(t)$, *are solved using either modal or direct-integration methods.*

"In this seminar report, a nonlinear time history analysis is performed on a ten storey RCC building frame considering time history of el centro earthquake 1940 using ETABS. The main parameters of the seismic analysis of structures are load carrying capacity, ductility, stiffness, damping and mass. The various response parameters like base shear, storey drift, storey displacements etc are calculated. The storey drift calculated is compared with the minimum requirement of storey drift as per IS 1893:2002.

Index Terms—Base Shear, Finite Element, Storey Drift, Roof displacement, Time history analysis, Response spectrum curve.

1. INTRODUCTION-

All real physical structures, when subjected to loads or displacements, behave dynamically. The additional inertia forces, from Newton's second law, are equal to the mass times the acceleration. If the loads or displacements are applied very slowly then the inertia forces can be neglected and a static load analysis can be justified. Hence, dynamic analysis is a simple extension of static analysis

In addition, all real structures potentially have an infinite number of displacements. Therefore, the most critical phase of a structural analysis is to create a computer model, with a finite number of mass less members and a finite number of node (joint) displacements, that will simulate the behavior of the real structure. The mass of a structural system, which can be accurately estimated, is lumped at the nodes. Also, for linear elastic structures the stiffness properties of the members, with the aid of experimental data, can be approximated with a high degree of confidence.

However, the dynamic loading, energy dissipation properties and boundary (foundation) conditions for many structures are difficult to estimate. This is always true for the cases of seismic input or wind loads.

"The current version of the IS: 1893 - 2016 states that linear dynamic analysis shall be performed to obtain the design lateral force (design seismic base shear, and its distribution to the different levels along the height of the building, and to the various lateral load resisting element) for all building, other than regular building lower than 15m in seismic zone II. Practically all multistoried buildings be analyzed as three-dimensional systems. This is due to the fact that the buildings have generally irregularities in plan or elevation or in both.

As per Indian standard code 1893 (part 1) : 2016 Time history method shall be based on an appropriate ground motion (preferably compatible with the design acceleration spectrum in the desired range of natural period) and shall be performed using accepted principles of the earthquake structural dynamics.

2. DIFFERNENT METHOD OF THE SEISMIC ANALYSIS-

Seismic analysis is a subset of structural analysis and is the calculation of the response of a building (or non building) structure to earthquakes. It is part of the process of structural design, earthquake engineering or structural assessment and retrofit (see structural engineering) in regions where earthquakes are prevalent.

Based on the type of external action and behavior of structure, the analysis can be further classified as:

(1).Linear Static Analysis, (2).Nonlinear Static Analysis, (3).Linear Dynamic Analysis and (4). Nonlinear Dynamic Analysis.

Linear static analysis or equivalent static method can be used for regular structure with limited height. Linear dynamic analysis can be performed by response spectrum method. The significant difference between linear static and linear dynamic analysis is the level of the forces and their distribution along the height of structure. Nonlinear static analysis is an improvement over linear static or dynamic analysis in the sense that it allows inelastic behavior of structure. A nonlinear dynamic analysis is the only method to describe the actual behavior of a structure during an earthquake. The method is based on the direct numerical integration of the differential equations of motion by considering the elasto-plastic deformation of the structural element.

2.1 Equivalent Static Analysis-

This procedure does not require dynamic analysis, however, it account for the dynamics of building in an approximate manner. The static method is the simplest one-it requires less computational efforts and is based on formulate given in the code of practice. First, the design base shear is computed for the whole building, and it is then distributed along the height of the building. The lateral forces at each floor levels thus obtained are distributed to individuals lateral load resisting elements (Duggal, 2010).

2.2 Nonlinear Static Analysis-

It is practical method in which analysis is carried out under permanent vertical loads and gradually increasing lateral loads to estimate deformation and damage pattern of structure. Non linear static analysis is the method of seismic analysis in which behavior of the structure is characterized by capacity curve that represents the relation between the base shear force and the displacement of the roof. It is also known as Pushover Analysis.

2.3 Linear Dynamic Analysis-

Response spectrum method is the linear dynamic analysis method. In that method the peak response of structure during an earthquake is obtained directly from the earthquake response, but this is quite accurate for structural design applications (Duggal, 2010).

2.4 Nonlinear Dynamic Analysis-

It is known as Time history analysis. It is an important technique for structural seismic analysis especially when the evaluated structural response is nonlinear. To perform such an analysis, a representative earthquake time history is required for a structure being evaluated. Time history analysis is a step-by step analysis of the dynamic response of a structure to a specified loading that may vary with time. Time history analysis is used to determine the seismic response of a structure under dynamic loading of representative earthquake (Wilkinson and Hiley, 2006).

3. TIME FUNCTION-

CSI Software handles the initial conditions of a time function differently for linear and nonlinear time-history load cases.

Linear cases always start from zero, therefore the corresponding time function must also start from zero.

- **Nonlinear cases** may either start from zero or may continue from a previous case. When starting from zero, the time function is simply defined to start with a zero value. When analysis continues from a previous case, it is assumed that the time function also continues relative to its starting value. A long record may be broken into multiple sequential analyses which use a single function with arrival times. This prevents the need to create multiple modified functions.

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Here for analysis of the ten storey RC building the time function of linear case has been chosen for the time history analysis.

4. MODAL ANALYSIS METHOD-

Fast Nonlinear Analysis (FNA) is a modal analysis method useful for the static or dynamic evaluation of linear or nonlinear structural systems. Because of its computationally efficient formulation, FNA is well-suite for time-history analysis, and often recommended over direct-integration applications. During dynamic-nonlinear FNA application, analytical models should-

1-Be primarily linear-elastic.
2-Have a limited number of predefined nonlinear members.
3-Lump nonlinear behavior within link objects.
Here for analysis of the ten storey RC building the time function of linear case and Fast non linear modal analysis method has been chosen for the time history analysis.

5. GROUND MOTION EXCITATION-

Selecting the seismic loading for design and/or assessment purposes is not an easy task due to the uncertainties involved in the very nature of seismic excitations. One possible approach for the treatment of the seismic loading is to assume that the structure is subjected to a set of records that are more likely to occur in the region where the structure is located.

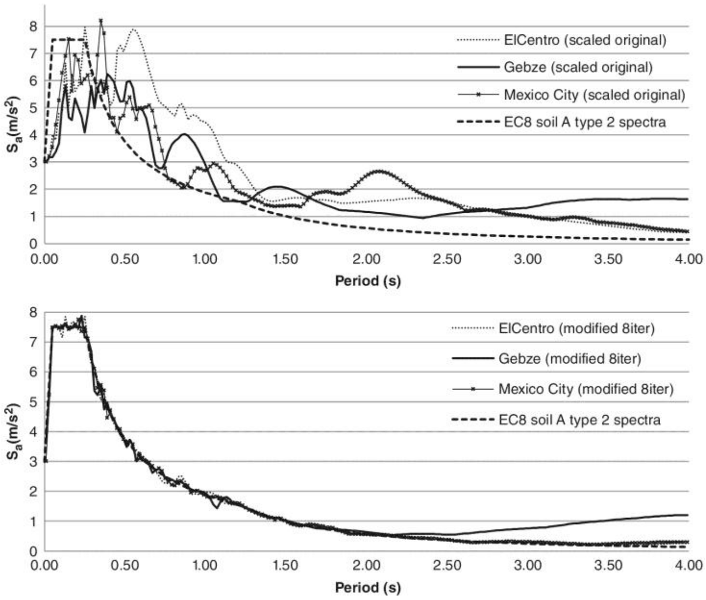


FIGURE 1- ARTIFICIAL ACCELEROGRAM GENERATOR

This figure shows the example of an artificial accelerogram generator code written in software with iteration program.

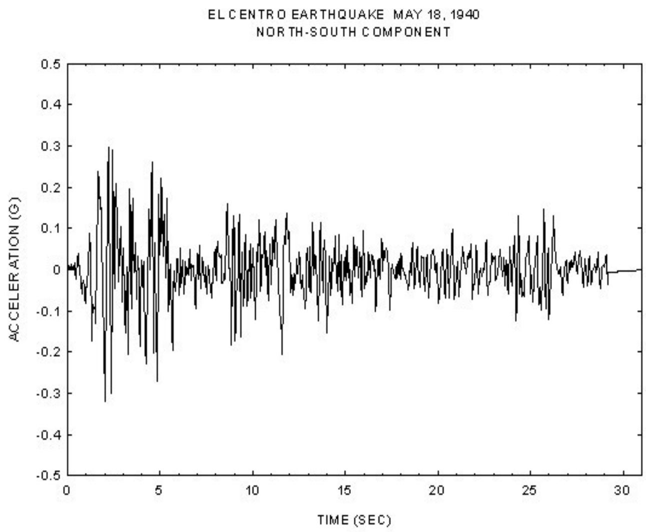


FIGURE 2- TIMEV/SACCELERATION OF ELCENTRO

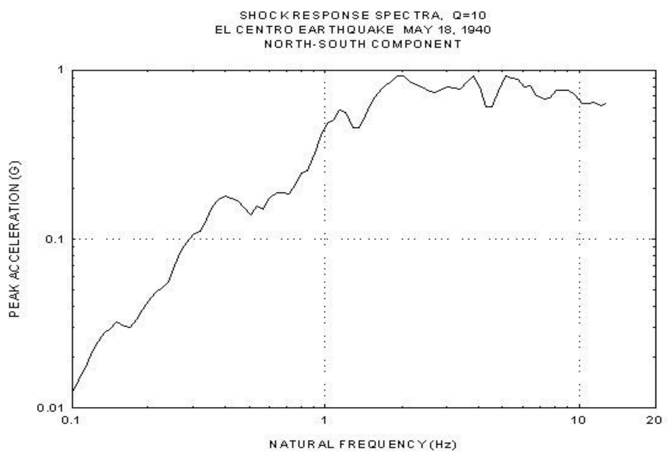


FIGURE 3- NATURAL FREUENCY VS PEAK ACC

6. STRUCTURAL MODELLING AND ANALYSIS-

The finite element analysis-FEM based software ETABS is used to create 3D model and run all analyses. The software is able to predict the geometric nonlinear behavior of space frames under static or dynamic loadings, taking into account both geometric nonlinearity and material inelasticity. In this report, a nonlinear time history analysis will be performed on a multi storey RCC building frame considering time history of **EL CENTRO EARTHQUAKE 1940**.

6.1- Description Of Elcentro Earthquake-

El Centro 1940-05-19 04:36:41 UTC
STATION - EL-CENTRO ARRAY

Region: California
Latitude: 32.7601
Longitude: -115.4162
Depth: 8.8 km
Mechanism: Strike-slip

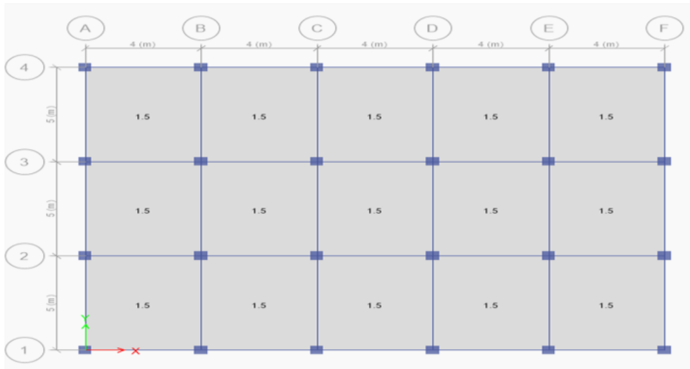
6.2-PROBLEM STATEMENT-

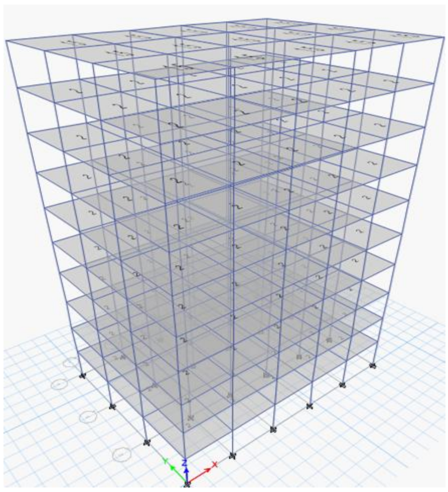
A 10 storey RCC masonry infilled RCC building have
Floor to Floor height- 3.1 m.
LL on Typical floors 2 KN/m2 & SIDL or FF- 1kN/m2
Live Load on Terrace - 1.5 KN/m2 & SIDL- 2kN/m2
Column size - 0.45 m X 0.45 m
Beams size - 0.23 m X 0.45 m
Slab Thickness - 0.150 m;
Brick wall thickness -0.23m
Density of concrete- 25 kN/m3
Density of brick wall- 20kN/m3
Load intensity for 10mm thick mortar- 0.21kN/m2
Height of parapet wall-1m
For structural analysis moment of inertia may be taken as - 70% of gross moment of inertia of column and 35% of gross moment of inertia of beam (In RC and masonry structures).
Number of modes considered initially - 12 nos.
Circular frequency, ω (rad/sec) = $2\pi T$
Eigen value = ω2
Frequency (cycle/sec) = $1/T$
Use M25 concrete and Fe415 steel.

6.2- LOAD CALCULATION -

Dead load (self wt.) of slab= 0.15x1x25= **3.75kN/m**
Wall load intensity= 0.23 x (3.1-0.45) x 20= **12.19kN/m**
Parapet wall load intensity= 0.23 x 1 x 20= **4.6kN/m**
Superimposed dead load or floor finish- 0.21x5=**1.05kN/m2**.

Below figure show the mode of the structure-





7. ANALYSIS RESULTS-

7.1- BASE REACTION-

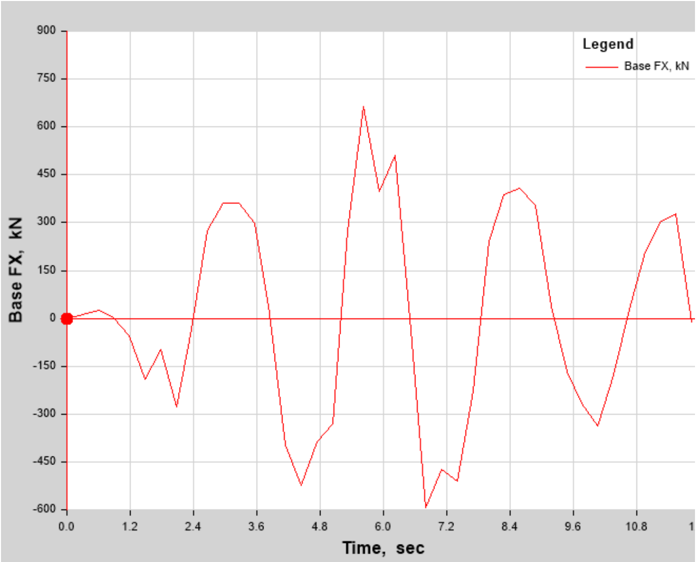


FIGURE 4- BASE REACTION FX IN X-DIR

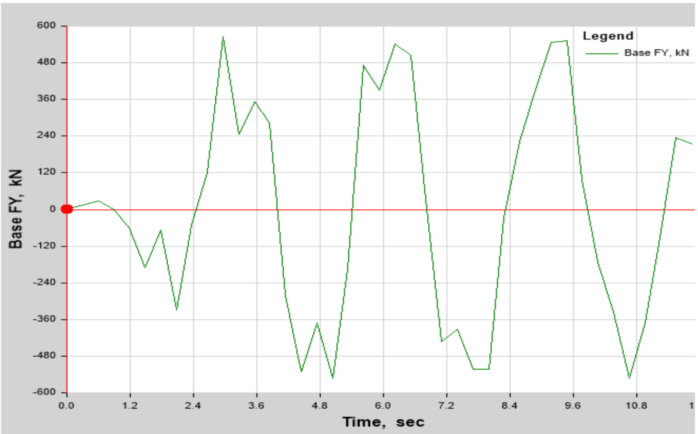


FIGURE 5- BASE REACTION FX IN Y-DIR

TIME SEC	BASE- FX
0.296	12.8005
2.664	273.5706
3.256	359.866
3.552	298.7083
5.328	275.788
5.624	664.143
6.216	512.0627
7.992	240.5218
8.288	387.0604
11.248	300.2585

TABLE: Time History Plot

Time	Base FY
sec	kN
0.592	27.1071
0.888	0.4892
5.624	472.3558
5.92	389.9112
6.216	540.7972
6.512	505.5179
8.88	393.5661
9.176	547.1206
9.472	551.5677
11.84	213.5987

TIME	MAX BASE REACTION
5.624 sec	664.143 kN in X-dir
2.96 sec	565.974 kN in y-dir

7.2 MODAL PERIODS AND FREUENCY-

TOTAL NOS OF MODES- changed to 15 (modal participation factor of mode k contributes to the overall vibration of the structure under horizontal and vertical ground motion. since the amplitude of 95 percent mode shape can be scale arbitrarily, the value of this factor depends on the scaling used for mode shapes

FUNDAMENTAL NATURAL PERIOD - it is the first (longest)modal time period of vibration in our case the fundamental natural period is found out to be 2.97 sec and the frequency of 0.377 cyc/sec

TABLE: Modal Periods And Frequencies

Case	Mode	Period	Frequency	CircFreq
		sec	cyc/sec	rad/sec
Modal	1	2.97	0.337	2.1156
Modal	2	2.644	0.378	2.3766
Modal	3	2.489	0.402	2.5245
Modal	4	0.944	1.06	6.6582
Modal	5	0.848	1.179	7.4074
Modal	6	0.797	1.255	7.8876
Modal	7	0.529	1.889	11.8707
Modal	8	0.483	2.07	13.0053
Modal	9	0.452	2.211	13.8947
Modal	10	0.347	2.885	18.1252
Modal	11	0.322	3.103	19.4982

Modal	12	0.3	3.338	20.9726
Modal	13	0.242	4.136	25.9899
Modal	14	0.228	4.388	27.5684
Modal	15	0.211	4.744	29.8094

7.3-STORY DRIFTS-

Story	X-Dir	HEIGHT	STORY DRIFT
Story10	0.000512	2950	1.5104
Story9	0.000836	2950	2.4662
Story8	0.001158	2950	3.4161
Story7	0.001461	2950	4.30995
Story6	0.00171	2950	5.0445
Story5	0.001757	2950	5.18315
Story4	0.001779	2950	5.24805
Story3	0.001638	2950	4.8321
Story2	0.001567	2950	4.62265
Story1	0.000926	2950	2.7317
Base	0	2950	0

Story	Y-Dir	HEIGHT	STORY DRIFT
Story10	0.000779	2950	2.29805
Story9	0.001296	2950	3.8232
Story8	0.001754	2950	5.1743
Story7	0.002141	2950	6.31595
Story6	0.002377	2950	7.01215
Story5	0.002234	2950	6.5903
Story4	0.002025	2950	5.97375
Story3	0.002145	2950	6.32775
Story2	0.001965	2950	5.79675
Story1	0.001048	2950	3.0916
Base	0	2950	0

This story drift is due to the load case- of time history analysis not for any type of individual or load combinations.

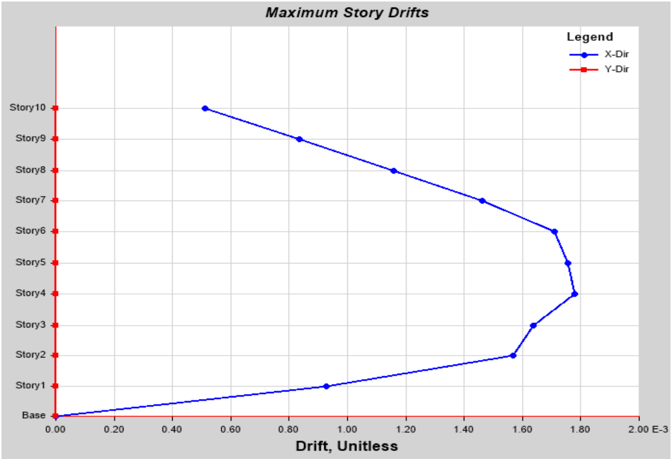


Figure6 - Maximum Story Drifts In X- Dir

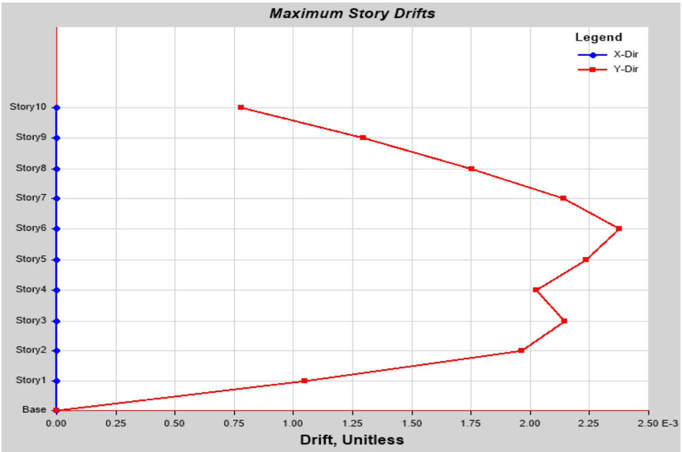


Figure7 - Maximum Story Drifts In Y- Dir

7.4-RESPONSE SPECTRUM CURVES-

Response-spectrum analysis provides insight into dynamic behavior 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 envelope response spectra such that a smooth curve represents the peak response for each realization of structural period.

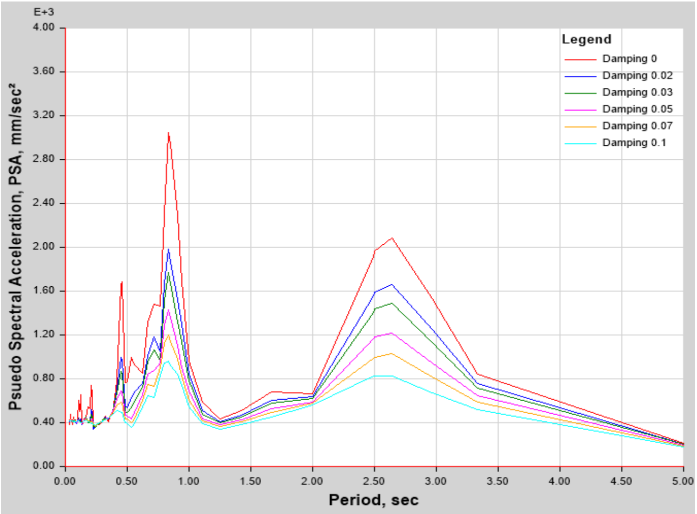


Figure8 - Response In X- Dir

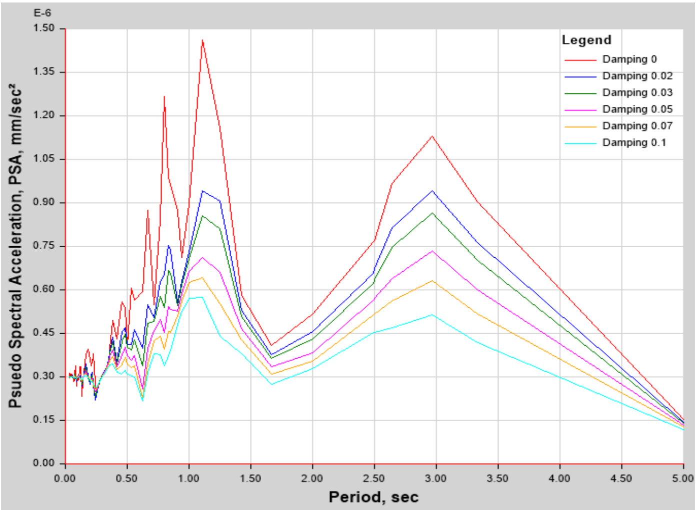


Figure9 - Response In Y- Dir

RESULTS FROM RESPONSE SPECTRUM CURVES-

Time history case -	*X*
Story	10
Joint lable	1
Response direction	*X*
X- Axis	period in sec
Y-axis	PSA in mm/sec2
Max Snapped To (0.833, 1432.84)	
TIME HISTORY CASE -	**Y**
Story	10
Joint lable	1
Response direction	Y
X- Axis	period in sec
Y-axis	PSA in mm/sec2
Max Snapped To (0.7142, 1174.7142)	

7.5- COMBINED STOREY DISPLACEMENT-

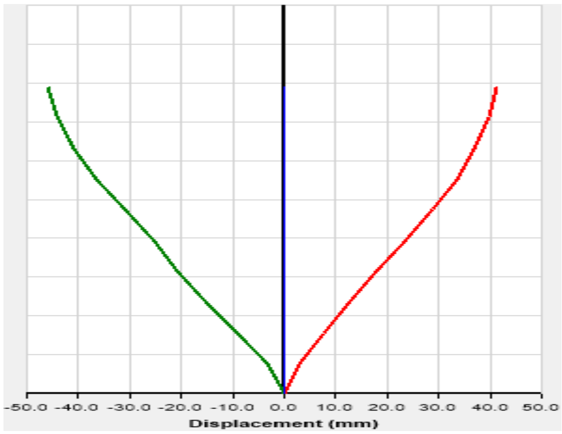


Figure10 - Displacement In X- Dir

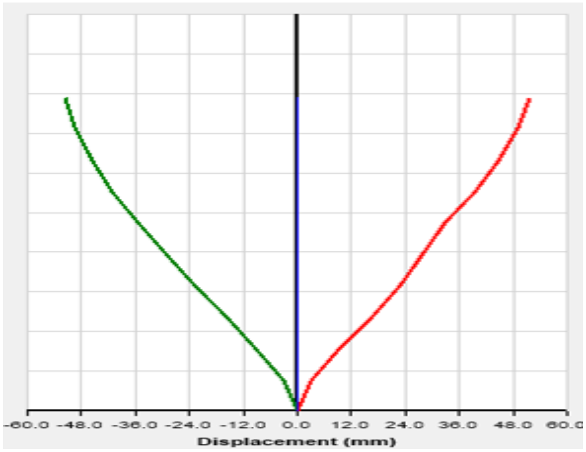


Figure11 - Displacement In Y- Dir

8. CONCLUSIONS-

1. Results from the base reaction conclude that the base reaction increases with the time in sec from the time history plot we can see that the base reaction is max at 5.624 sec with 664.147 kN at x- direction where as the max base reaction obtain on in y direction is at 3.5sec with 565.974 kN. both reaction are comparably very high and should be designed appropriately.
2. The numbers of mode shapes considered are 12 (initially)and then change to 15 after a trial modal analysis to match the MPM ratio and for each mode number the time period, frequency and eigen values are mentioned above.

3. The variation of base shear in X and Y direction with respect to time history of el centro earthquake ground motion is plotted.
4. Similarly the variation of storey drift in X and Y direction with respect to time history is also plotted. which shows that the max drift obtain in x-dir is at storey 4 of 5.24mm and that of in y- direction is at storey 6 of 7.01mm
5. Psuedo spectral acceleration is max snapped to (0.833, 1432.84) for response in X-direction and max snapped to (0.7142, 1174.7142) for response in Y- direction. where(X,Y) is (seconds, acceleration)
6. Maximum displacement is 42.34mm & 53.15mm at X and Y direction respectively as shown in figure 10 and 11.

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