

SEISMIC ANALYSIS AND CONTROL OF R.C.C. CHIMNEY UNDER NEARFIELD AND FARFIELD EARTHQUAKE

Alqama Hasan¹, Dr. Misbah Danish Sabri², Gyan Singh (Structure Consultant)

¹Student of M. Tech in Structure & Foundation, Al-Falah University, Haryana, India

²Professor and Advisor, Civil Engineering Dept., Al-Falah University, Haryana, India

Abstract - A structure may be subjected to earthquake loading several times during its life span. Major or minor damage may occur due to effect of vibration of earthquake. So it is very important to analyze the effect of earthquake on the structure and to control or reduce the effect of earthquake. There are so many methods of seismic analysis of structure but nonlinear time history analysis gives the actual behavior of the structure under earthquake loading. In this thesis R.C.C. chimney of circular section is analyzed on the basis of finite element modelling using SAP2000 software. Nonlinear time history analysis of R.C.C. chimney is carried out considering two different earthquakes ground motions. Elcentro near field and Landers Baker far field earthquake records are taken as input ground motion for the nonlinear time history analysis of R.C.C. chimney considering base as fixed and different seismic responses such as maximum base shear, maximum base moment, maximum top displacement and time period of the chimney are studied. Here base isolation technique is also applied to control the responses by providing the laminated rubber bearing as isolator. Further the responses of base fixed and base isolated chimney are studied, compared and percentages of response control are found out for a particular type of isolator.

From the results it is observed that seismic responses under Elcentro near field earthquake are found very high as compared to Landers Baker far field earthquake and it is also observed that base isolation significantly reduces the seismic responses under strong ground motion, because of the decoupling of superstructure from the earthquake ground motion by introducing a flexible interface between the foundation and the base of structure.

Key Words: Non-linear time history analysis, Earthquake effects due to near and far field and dynamic analysis for the chimney structures.

1. INTRODUCTION

Chimneys are tall, slender and generally with circular cross-sections. Different construction materials, such as concrete, steel or masonry, are used to build chimneys. Chimneys are symbol of industrial growth of any country. In Recent years there has been increased demand for tall chimneys due setting up several large thermal power stations in the country. In view of stricter control on air pollution the trend is towards constructing taller chimneys. Now-days many chimneys in the range of 220m height have been constructed in our country. In the USA several chimneys in the range of

380m already exist and this trend towards constructing taller chimneys will continue. Construction of such tall chimneys have been possible with better understanding of loads acting on them and of the structural behavior, above all with the utilization of modern construction plant equipment and techniques such as slip form.

Many of the existing industrial chimneys in many parts of the world are constructed of unreinforced masonry. These structures are found to be vulnerable to damage during earthquakes. It has been documented that these types of industrial chimneys may fail at either of two different locations. These structures have experienced extensive cracking or complete failure either at or near its base or at the top third of its height (China Academy of Building Research, 1986). Several cases of failure at the top third of the chimney's height were observed in chimneys located in city of Tangshan, China during the earthquake event of July 28, 1976 (Shu-quan, 1981). Chimneys those were located in city of Beijing, China, experienced failure at their bases. Reinforced Concrete has been the most favoured material for the construction of chimney since it has the advantage to resist the wind load and other forces acting on them, as a self-standing structure. With the experience gained in the chimney design and construction over the years the country has been able to meet the present day challenges of designing and constructing other similar tall structures such as television towers, urea prilling towers, silos etc. One of the early specifications for the design of tall chimneys was prepared by American Concrete Institute in 1936 which has undergone revisions subsequently and the latest being 307-791. The Indian code of practice for the design of RCC chimney IS: 49982 was first published in 1964 which has since been revised in 1975.

2. Objective

Based on the literature review studied, the objective of the present study is defined as follows:

1. The main objective of this study is to analyse the R.C.C. chimney under earthquake loading considering the base of chimney as fixed and isolated.
2. To find Time Periods of different modes and mode shapes.
3. To plot the time history of base shear, base moment and top displacement thus find the maximum base shear,

maximum base moment and lateral deflection of the top of the chimney.

4. To find the variation of shell stress and lateral displacement along the height of the chimney.

5. To control or reduce the responses using laminated rubber bearing as isolators.

3. Seismic Analysis and I.S. Code Provisions

3.1 General

Seismic analysis is related to calculation of the response of a building or other structures under earthquakes. It is a part of the process of structural design which includes earthquake engineering or structural assessment and retrofit in regions where earthquakes are prevalent. During earthquake many of the buildings collapse due to lack of understanding of the inelastic behaviour of structure. Elastic analysis gives only elastic capacity of the structure and indicates where the first yielding occurs. It cannot give any information about redistribution of forces and moments and failure mechanism.

For study of inelastic behaviour of structure nonlinear analysis is necessary. The development of rational methodology that is applicable to the seismic design of new structures using available ground motion information and engineering knowledge, and yet is flexible enough to permit the incorporation of new technology as it becomes available has been supported for sometimes now. This is the focus of several major research and development efforts throughout the world. In majority of cases nonlinear analysis is used.

3.2 Method for Linear Static Analysis

This approach defines a series of forces acting on a building to represent the effect of earthquake ground motion, typically defined by a seismic design response spectrum. It assumes that the building responds in its fundamental mode. For this to be true, the building must be low-rise and must not twist significantly when the ground moves.

The response is read from a design response spectrum, given the natural frequency of the building. The applicability of this method is extended in many building codes by applying factors to account for higher buildings with some higher modes, and for low levels of twisting. To account for effects due to "yielding" of the structure, many codes apply modification factors that reduce the design forces.

3.3 Methods for Linear Dynamic Analysis

3.3.1 Linear Dynamic Analysis

[1] Static procedures are appropriate when higher mode effects are not significant. This is generally true for short, regular buildings. Therefore, for tall buildings,

buildings with torsional irregularities, or non-orthogonal systems, a dynamic procedure is required.

In the linear dynamic procedure, the building is modelled as a multi degree of freedom (MDOF) system with a linear elastic stiffness matrix and an equivalent viscous damping matrix. The seismic input is modelled using either modal spectral analysis or time history analysis but in both cases, the corresponding internal forces and displacements are determined using linear elastic analysis. The advantage of these linear dynamic procedures with respect to linear static procedures is that higher modes can be considered. However, they are based on linear elastic response and hence the applicability decreases with increasing nonlinear behaviour, which is approximated by global force reduction factors.

In linear dynamic analysis, the response of the structure to ground motion is calculated in the time domain, and all phase information is therefore maintained. Only linear properties are assumed. The analytical method can use modal decomposition as a means of reducing the degrees of freedom in the analysis.

3.3.2 Linear Time-History Analysis

Time-history analysis is a step-by-step analysis of the dynamical response of a structure to a specified loading that may vary with time. The analysis may be linear or non linear.

3.4 Methods for Nonlinear Analysis

The nonlinear static procedures constitute an inelastic analysis that considers what happens to buildings after they begin to crack and yield in response to realistic earthquake motion. This approach differs from traditional static linear procedure that reduces seismic forces to levels that allow designing buildings under the assumption that they remain undamaged. Although unrealistic and potentially misleading, this simplistic approach works well for new buildings and for regular existing buildings.

3.4.1 Secant Method

When the analysis of building is done with the Secant method, a global elastic model of the structure is constructed. Special stiffness values are calculated for the modelled elements and components. The global elastic model is analysed using elastic response spectrum analysis. The ground motion used in the analysis is either a code specified 5 % damped response spectrum or the 5 % site specified spectrum. In general, the response spectrum analysis will predict a different displacement pattern than originally assumed. At this point, iteration begins. The pushover curves are used to select a new set of element secant stiffness based on the displacements predicted by the global analysis. The global elastic model is modified with the new secant stiffness, and the response spectrum analysis is

repeated. This process continues until the displacements predicted by the computer model reasonably match the displacements used to calculate the secant stiffness, at which point the analysis has predicted the earthquake demand.

The principal advantages of the secant method are that it accounts for three dimensional effects including torsion and multi-direction loading and that it accounts for higher mode effects. The main disadvantage of the approach is that it can be somewhat more time consuming than other static nonlinear procedures.

3.4.2 Method of Pushover Analysis

Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. With the increase in the magnitude of the loading, weak links and failure modes of the structure are found. The loading is monotonic with the effects of the cyclic behaviour and load reversals being estimated by using a modified monotonic force deformation criteria and with damping approximations. Static pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the structure and it promises to be a useful and effective tool for detailed performance evaluation of building. The pushover analysis can be performed either under load control or displacement control as mentioned below.

1. Load Control: It is used when the load is known (such as gravity load) and the structure is expected to be able to support the full magnitude of the load which is applied in steps.

2. Displacement Control: In this method, the magnitude of the load combination is increased or decreased as necessary until the control displacement reaches a predefined value. It is used when specified drifts are sought, magnitude of the applied load is not known in advance, structure can be expected to lose strength or become unstable or when displacement occurring in the design earthquake is known.

3.4.3 Nonlinear Time History Analysis Method

Some buildings may be too complex to rely on the nonlinear static procedure. Those cases may require time history analysis of the nonlinear behaviour of the structure during analysis for a particular example of earthquake. The kinds of the buildings that may require this specialized analysis are highly irregular or complicated.

This method is performed using time histories prepared according to the actual ground motions recorded. The requirements for the mathematical model for time history analysis are identical to those developed for response spectrum analysis. The damping matrix associated with the

mathematical model shall reflect the damping inherent in the structure deformation levels less than the yield deformation.

Response parameters shall be calculated for each time-history analysis. If three time-history analysis are performed, the maximum response of the parameter of interest shall be used for design. If seven or more pairs of horizontal ground motion records are used for time-history analysis, the average response of the parameter of interest may be used for design.

3.4.4 Fast Non-linear Analysis Method

The response of real structures when subjected to a large dynamic input often involves significant nonlinear behaviour which includes the effects of large displacements and/or nonlinear material properties. The use of geometric stiffness and P-Delta analyses includes the effects of first order large displacements. If the axial forces in the members remain relatively constant during the application of lateral dynamic displacements, many structures can be solved directly without iteration.

The more complicated problem associated with large displacements, which cause large strains in all members of the structure, requires a tremendous amount of computational effort and computer time to obtain a solution. Fortunately, large strains very seldom occur in typical civil engineering structures made from steel and concrete materials. However, certain types of large strains, such as those in rubber base isolators and gap elements, can be treated as a lumped nonlinear element using the Fast Nonlinear Analysis (FNA) method.

The more common type of nonlinear behaviour is when the material stress strain, or force-deformation, relationship is nonlinear. This is because of the modern design philosophy that "a well-designed structure should have a limited number of members which require ductility and that the failure mechanism be clearly defined." Such an approach minimizes the cost of repair after a major earthquake.

3.5 Modal Analysis

Modal analysis is used to determine the vibration modes of a structure. These modes are useful to understand the behaviour of the structure. They can also be used as the basis for modal superposition in response spectrum and modal time-history analysis cases.

3.6 Response Spectrum Method

This approach permits the multiple modes of response of a building to be taken into account. This is required in many building codes for all except for very simple structures. The response of a structure can be defined as a combination of many special shapes (i.e. modes) that in a vibrating string

correspond to the "harmonics". Computer analysis can be used to determine these modes for a structure.

For each mode, a response is read from the design spectrum, based on the modal frequency and the modal mass, and they are then combined to provide an estimate of the total response of the structure. Modal combination methods are:

1. Absolute Sum Method (ASM) combines the modal results by taking the sum of their absolute values.

2. Square Root of the Sum of the Squares (SRSS) combines the modal results by taking the square root of the sum of their squares.

3. Complete Quadratic Combination (CQC) method takes into account the statistical coupling between closely spaced modes caused by modal damping and also it is a method that is an improvement on SRSS for closely spaced modes.

It should be noted that the result of a response spectrum analysis using the response spectrum from a ground motion is typically different from that which would be calculated directly from a linear dynamic analysis using that ground motion directly, since phase information is lost in the process of generating the response spectrum.

In cases where structures are either too irregular, too tall or of significance to a community in disaster response, the response spectrum approach is no longer appropriate, and more complex analysis is often required, such as non-linear static or dynamic analysis.

3.7 I.S. Code Provisions

Stack like structures are those in which the mass and stiffness is more or less uniformly distributed along the height. Cantilever structures like reinforced or prestressed cement concrete electric poles; reinforced concrete brick and steel chimneys (including multiflue chimneys), ventilation stacks and refinery vessels are examples of such structures.

[2] 3.7.1 Time Period of Vibration

Time period of vibration, T of such structures when fixed at base, shall be calculated using either of the following two formulae given. The formulae given by eq.(3.1), is more accurate. Only one of these two formulae should be used for design. Time period of structure, if available, through vibration measurement on similar structure and foundation soil condition can also be adopted.

The fundamental time period for stack like structures, 'T' is given by:

$$T = C_T \sqrt{\frac{W_t H}{E_s A g}} \dots (3.1)$$

Where,

C_T = Coefficient depending upon the slenderness ratio of the structure given in Table 3.1,

W_t = Total weight of the structure including weight of lining and contents above the base,

H = height of structure above the base,

E_s = Modulus of elasticity of material of the structural shell,

A = Area of cross-section at the base of the structural shell,

g = Acceleration due to gravity.

Note — this formula is only applicable to stack-like structure in which the mass and stiffness are more or less uniformly distributed along the height.

The fundamental time period, T of a stack like structure can be determined by Rayleigh's approximation for fundamental mode of vibration as follows:

$$T = \frac{2\pi}{\sqrt{g\delta}} \dots (3.2)$$

$$\delta = \frac{\sum_{i=1}^n W_i \delta_i}{\sum_{i=1}^n W_i \delta_i^2} \dots (3.3)$$

Where,

W_i = Weight lumped at i^{th} location with the weights applied simultaneously with the force applied horizontally,

δ_i = Lateral static deflection under its own lumped weight at i^{th} location (chimney weight lumped at 10 or more locations),

n = Number of locations of lumped weight

3.7.2 Damping

The damping factor to be used in determining S_a/g depends upon the material and type of construction of the structure and the strain level. The following damping factors are recommended as guidance for different materials for fixed base condition and are given in the Table 3.1.

Table 3.1: Material Damping Factors for Design Basis Earthquake

Sl No.	Material	For Earthquake	Design
1.	Steel	0.05	
2.	Reinforced Concrete	0.05	
3.	Brick masonry and plain concrete	0.07	

Table 3.2: Values of C_T and C_V

Sl No.	$K=h/r_e$	Coefficient C_T	Coefficient C_V
1.	5	14.4	1.02
2.	10	21.2	1.12
3.	15	29.6	1.19
4.	20	38.4	1.25
5.	25	47.2	1.30
6.	30	56.0	1.35
7.	35	65.0	1.39
8.	40	73.8	1.43
9.	45	82.8	1.47
10.	50 or more	1.8k	1.50

Notes
 1. k = slenderness ratio, and
 2. r_e = radius of gyration of the structural shell at the base section

3.7.3 Horizontal Seismic Force

Using the period T, as indicated in eq. (3.1), the horizontal seismic coefficient A_h shall be obtained from the spectrum given in Fig.A1 or A2 of appendix A. The equivalent static lateral loads shall be determined from design acceleration spectrum value A_h , calculated from the following equation using time period T from eq. (3.1) or eq. (3.2):

$$A_h = \frac{Z S_B I}{2 g R} \quad (3.4)$$

The horizontal earthquake force shall be assumed to act alone in one lateral direction at a time. The effects due to vertical component of earthquakes are generally small and can be ignored.

The vertical seismic coefficient, where applicable may be taken as 2/3 of horizontal seismic coefficient, unless evidence of factor larger than above is available.

The effect of earthquake and maximum wind on the structure shall not be considered simultaneously.

3.7.4 Design Shear Force and Moment

Either simplified method (That is equivalent static lateral force method) or the dynamic response spectrum model analysis method is recommended for calculating the seismic forces developed in such structures. Site spectra compatible time history analysis may also be carried out instead of response spectrum analysis.

The simplified method can be used for stack like structures. The design shear force V and design bending moment M for such a structures at a distance x from the top shall be calculated by the following formulae:

$$V=C_V A_h W_t \left\{ 0.4 \left(\frac{x}{H} \right)^{0.5} + 0.6 \left(\frac{x}{H} \right)^4 \right\} \dots(3.5)$$

$$M=A_h W_t h_{cg} \left\{ 1.1 \left(\frac{x}{H} \right)^{0.5} + 0.75 \left(\frac{x}{H} \right) + 0.9 \left(\frac{x}{H} \right)^4 \right\} \dots(3.6)$$

Where,

C_V = coefficient of shear depending on slenderness ratio k given in table

A_h = Design horizontal seismic coefficient determined in accordance with

W_t = total weight of the structure including weight of lining and contents above the base

h_{cg} = Height of c.g. of structure above the base

x= Distance of the section considered from the top

3.7.5 Deflection Criterion

The maximum lateral deflection at the top of a stack like structure under all service conditions, prior to the application of load factors shall not exceed the limits set forth by the following equation:

$$D_{max}=0.005H \dots(3.7)$$

Where,

D_{max} = Maximum lateral deflection, and

H=Height of structure above the base

4. Response Control System

4.1 General

Earthquake is a natural phenomenon of ground shaking caused by sudden release of energy in the earth's lithosphere (i.e. the crust plus part of upper mantle). This energy arises mainly from stresses built up during tectonic processes, which consist of interaction between the crust and the interior of the earth. As a result of excessive lateral displacement, the structures inability to dissipate energy will eventually lead to structural collapse. The debris of the collapsed structures has claimed many lives.

The control of structural vibrations produced by an earthquake can be done by various means such as modifying rigidities, masses, damping, or shape, and by providing passive or active counter forces. The selection of a particular type of vibration control device is governed by a number of factors which include efficiency, compactness and weight, capital cost, operating cost, maintenance requirements and safety.

4.2 Types of Base Isolation Devices

There are various types of bearings used in the base isolation systems, which vary according to their behavior and to the material they are made of. The most extensively used ones are the ones which belong to elastic systems class such as Rubber Bearing (RB), High Damping Natural Rubber Bearing (HDNR) and Steel Laminated Rubber Bearing (SLR), the ones belonging to elasto-plastic systems class such as Lead Rubber Bearing (LRB) and the ones belonging to kinematic systems class and friction pendulum systems class such as Friction Pendulum Bearing (FPB).

4.2.1 Rubber Bearings

These systems also have steel laminated rubber types and steel laminated rubber types with lead nucleus, along with the ones made of rubber and neoprene. The natural and artificial rubber bearings, which were used in bridge bearings, have later been developed and have been named elastomeric bearings. These bearings, which are used as seismic isolators, are widely used. The rubber laminated isolators are formed through vulcanization of thin steel plates to rubber plates (Fig.3.1). The more developed of those are laminated rubber types with lead nucleus. Lead Laminated Rubber Bearing systems are constituted by steel/rubber laminated layers with a lead nucleus embedded in the middle, and they are highly developed seismic isolators (Fig.3.2).

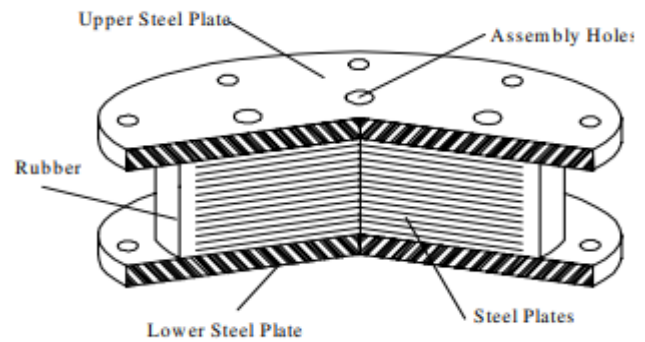


Fig. 4.1: Rubber laminated isolator

Another type of laminated rubber isolators allowing more lateral displacements are Slider Laminated Rubber Isolators. In these types, the laminated rubber cylindrical mass is surrounded by a sliding plate. Around the sliding rubber isolator, there is a steel stopper with a circular plan placed in such a way to allow displacements of certain size. Thus, in small seismic motions, the vibrations are damped through the deformation of laminated rubber, in larger motions, the structure is allowed to make a larger horizontal move through the sliding of the sliding plate.

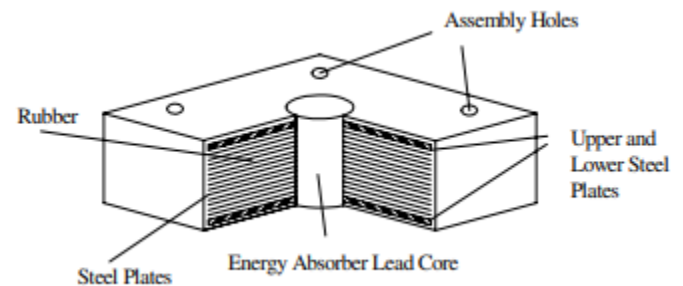


Fig. 4.2: Lead laminated rubber bearing

These types of bearings show a vertically rigid, horizontally flexible behavior. These bearings convey the vertical compounds of earthquake forces relatively to the structure they isolate the structure from the horizontal compounds under seismic loads. They are suitable for low-rise, rigid or pre-stressed buildings. In these types of buildings, use of rubber bearings is very useful. Base isolators are placed to balance the center of mass and center of rigidity. Thus, the negative effects of the irregularly designed structural system are eliminated. These bearings carry pressure loads at large amounts and accompany the movement in one or more direction in sliding different from the mechanical apparatus. Since rubber has a low shear modulus, the torsion freedom of rubber is decreased through placing steel laminates inside, and shear rigidity is increased a lot through these laminates. These bearings are very much resistant to environmental effects and long lasting.

Elastomeric bearings cannot resist to tensile stress formed by overturning moments. Equipment to resist tensile forces can be placed to this isolator to enable the rigidity of its structure. Elastomeric bearings can Steel Plates Lower Steel Plate Assembly Holes Upper Steel Plate Rubber Assembly Holes Upper and Lower Steel Plates Energy Absorber Lead Core Steel Plates Rubber be made of low or highly damp rubber.

4.2.2 Friction pendulum bearings

Friction pendulum systems are the most extensively used kinematic systems especially in base isolation. Pendulum system consists of a steel globe placed in two concave curved surface of steel (Fig.4.3) or a cylindrical member with global contact surfaces. In these parts special metals are used (Fig.4.4).

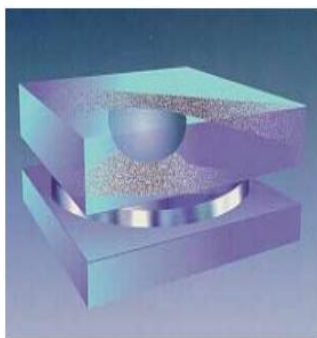


Fig.4.3: Cross section of a friction pendulum bearing Fig.4.4:Detail of friction pendulum bearing

These bearings, which have all the benefits of rubber bearings, through a bearing member which can slide on the global concave surface, it damps the energy because it assumes a position elevating the building during a lateral motion, and decreases the effect of earthquakes a lot. These bearings can be used in buildings, in spanning and in heavy roof systems, and also, through mechanical properties of special metals in their structure, they can be used successfully in cold regions with danger of freezing.

Modelling

5.1 General Description of Selected Chimney

From the discussions in the previous section it is clear that top-to-base diameter ratio and height to base diameter ratio are the two important parameters that define the geometry of RCC chimney. Design code IS 6533 (Part 2): 1989, limits minimum base diameter as 1.6 times the top diameter of the chimney. This gives maximum limit of top-to-base diameter ratio as $1 / 1.6 = 0.625$. Also, as per IS 6533 (Part 2): 1989, the height-to-base diameter ratio as per the code limits to $30 / 1.6 = 18.75$ (for a maximum top-to-base diameter ratio of 0.625).

Here seismic responses of R.C.C. chimney of 100 m height by non linear time history analysis are found out and responses are controlled by using base isolation technique. Laminated rubber bearing are provided in isolated case and isolator is modelled as rubber isolator. The chimney model was considered to be fixed at the base of the chimney for the analysis.

Table 5.1: Description of Selected Chimney

Height of the chimney, H	100m
Outer Dia. at bottom, D	10m
Outer dia. at top, d	5m
Thickness of shell at bottom	0.6m
Thickness of shell at top	0.2m

5.2 Material Properties

The material used for construction of chimney is reinforced cement concrete with M30 grade of concrete and Fe 415 grade of steel. The stress strain relationship used is as per IS 456:2000. The basic properties of material used are as follows:

Modulus of elasticity of concrete, $E_c = 27386128 \text{ kN/m}^2$

Characteristic Compressive strength of concrete, $f_{ck} = 30000 \text{ kN/m}^2$

Poisson's ratio of concrete, $\mu_c = 0.2$

Modulus of elasticity of steel, $E_s = 2 \times 10^8 \text{ kN/m}^2$

Yield stress for steel, $f_y = 500000 \text{ kN/m}^2$

Poisson's ratio of steel, $\mu_s = 0.3$

Damping of material = 0.05

5.3 Seismic Property

Table 5.2: Seismic Property of Site

Zone	IV
Zone Factor	0.24
Importance Factor	1.5
Soil Type	II
Response Reduction Factor	3

5.4 Input Ground Motion Record

Table 5.3: Input Ground Motion Record

Earthquake	Station	Dist. (km)	Magnitude	PGA (g)
Elcentro (Nearfield)	Brawley Airport	43.15	6.53	0.32
Landers Baker (Farfield)	Baker Fire Station	87.94	7.3	0.102

5.5 Property of Laminated Rubber Bearing (LRB) Isolator

Table 5.4: Properties of Rubber Bearing Isolator

Vertical Stiffness of Isolator	1500000 kN/m
Horizontal Linear Stiffness	2000 kN/m
Horizontal Non Linear Stiffness	20000 kN/m
Yield Force	110 kN
Post yield stiffness Ratio	0.1
Damping	10%

Results and Discussion 6.1 General

In the present study R.C.C. chimney of circular section is analysed for two different earthquake motions by non linear time history analysis using SAP 2000 software, to calculate base shear, base moment, shell stresses and top displacement. Responses are controlled using laminated rubber bearing isolator, and effectiveness of base isolation system is found out by percentage reduction in responses. Greater the value of percentage reduction, more is the effectiveness of base isolation. Detail of results from this study is presented in tabular form as well as in graphical form.

6.2 Input Ground Motion

Elcentro and Lander baker earthquake records are used as input ground motion for non linear time history analysis. In this study the ground motion records have been extracted from PEER Strong Motion Database of Berkeley University.

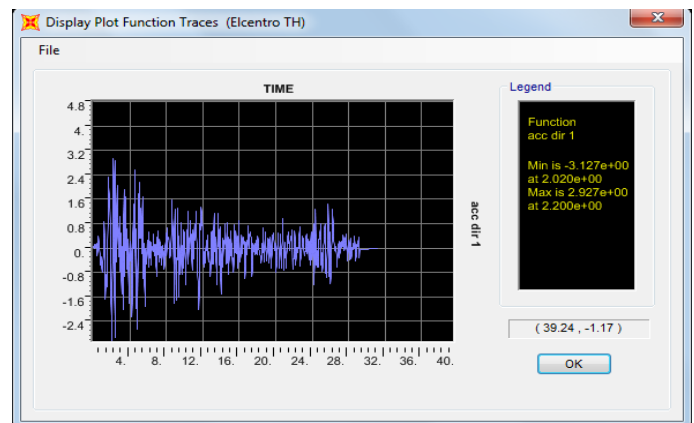


Fig. 6.1: Input Ground Motion of Elcentro Nearfield Earthquake

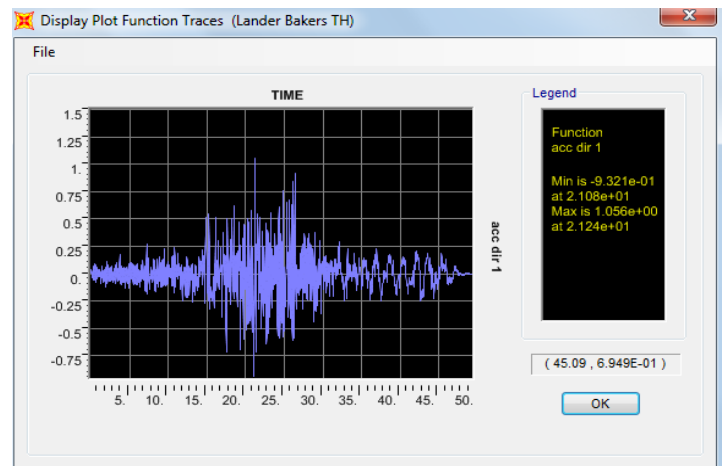


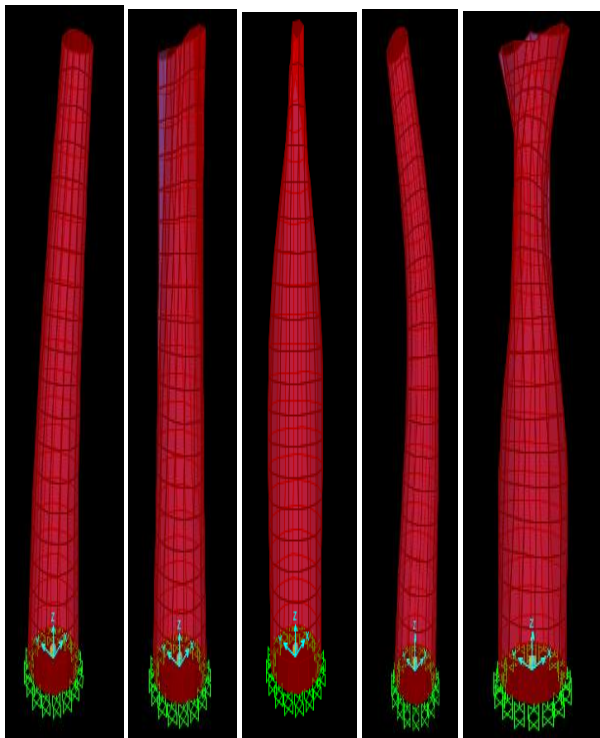
Fig. 6.2: Input Ground Motion of Landers Baker Farfield Earthquake

6.3 Results

For two different earthquake ground motions nearfield and farfield, considering the base of the chimney as fixed and as isolated seismic responses of the R.C.C. chimney are found out and comparison of different results is done in the tabular form as well as in graphical form. Finally effectiveness of base isolation is measured by percentage reduction in responses.

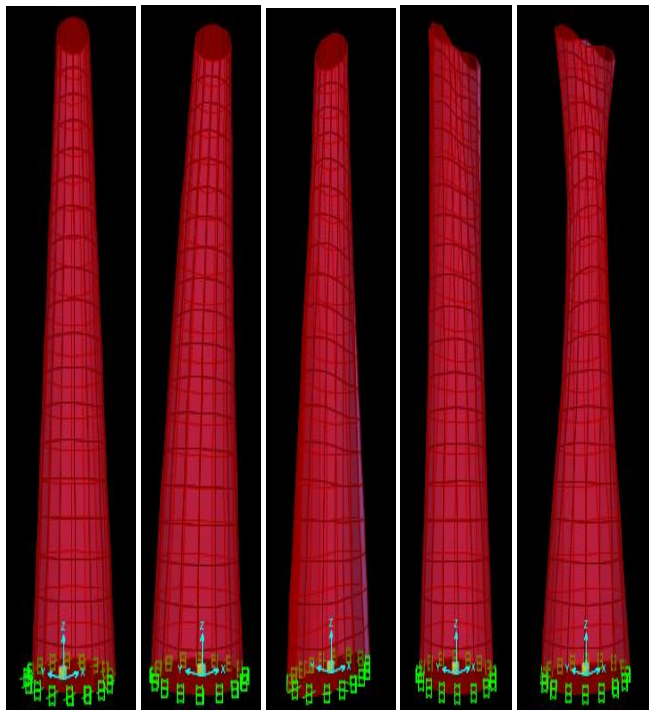
Table 6.1 Time Periods of Chimney

Mode No.	Time Period (sec.)	
	Base Chimney	Fixed Base Isolated Chimney
1.	0.550	2.654
2.	0.292	0.528
3.	0.248	0.329
4.	0.166	0.292
5.	0.162	0.237



Mode 1 Mode 2 Mode 3 Mode 4 Mode 5

Fig. 6.3: Mode Shapes of Base Fixed Chimney



Mode 1 Mode 2 Mode 3 Mode 4 Mode 5

Fig. 6.4: Mode Shapes of Base Isolated Chimney

Table 6.2: Maximum Base Shear of Chimney

Earthquake	Base Shear (kN)		Percentage Controlled in Base Shear
	Base Fixed Chimney	Base Isolated Chimney	
Elcentro	25750	3877	84.94
Lander Baker	4444	2042	54.05

Table 6.3: Maximum Base Moment of Chimney

Earthquake	Base Moment (kN-m)		Percentage Controlled in Base Moment
	Base Fixed Chimney	Base Isolated Chimney	
Elcentro	1251000	220700	82.36
Lander Baker	171800	100300	41.62

Table 6.4: Top Displacement of Chimney

Earthquake	Top Displacement (mm)		Percentage Controlled in Top Displacement
	Base Fixed Chimney	Base Isolated Chimney	
Elcentro	127.2	104.8	17.62
Lander Baker	17.39	28.1	No Control

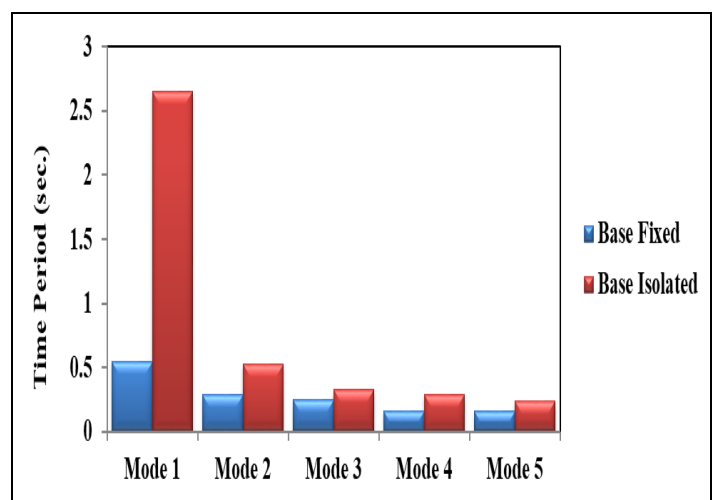


Fig. 6.5: Time Period

Fig. 6.5 illustrates the time period of base fixed and base isolated chimney. It can be seen that time period of base fixed chimney from 0.55 sec. in mode 1 to 0.16 sec. in mode

5. Similarly, in case of base isolated chimney it decreases from 2.65 sec. in mode 1 to 0.24 sec. in mode 5. This bar chart also shows that time periods of base isolated chimney are higher than those of base fixed chimney in all modes.

earthquake base moment reduces from 171800 kNm in base fixed chimney to 100300 kNm. This bar chart also shows that base moment in case of Elcentro earthquake is higher than that in case of Landers baker earthquake.

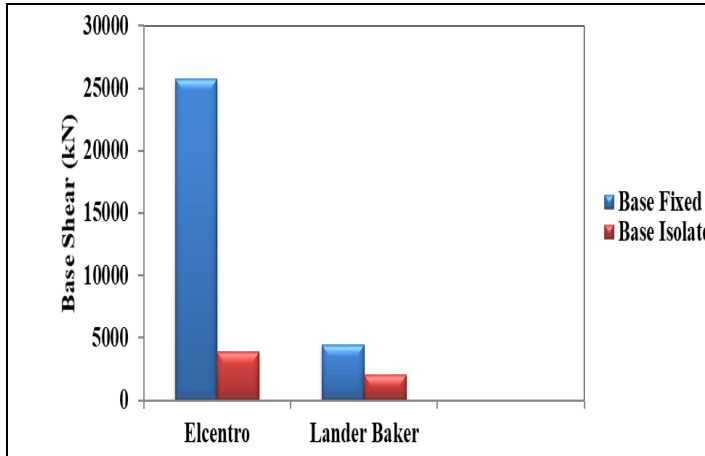


Fig. 6.6: Maximum Base Shear

Fig. 6.6 shows the comparison of max. base shear of base fixed chimney and base isolated chimney subjected to Elcentro nearfield and Landers baker farfield earthquake. It can be seen max. base shear under Elcentro ground motion reduces from 25750 kN in base fixed chimney to 3877 kN in base isolated chimney. In case of Landers baker earthquake base shear reduces from 4444 kN in base fixed chimney to 2042 kN in base isolated chimney. This bar chart also shows that base shear in case of nearfield earthquake is higher than that in case of farfield earthquake.

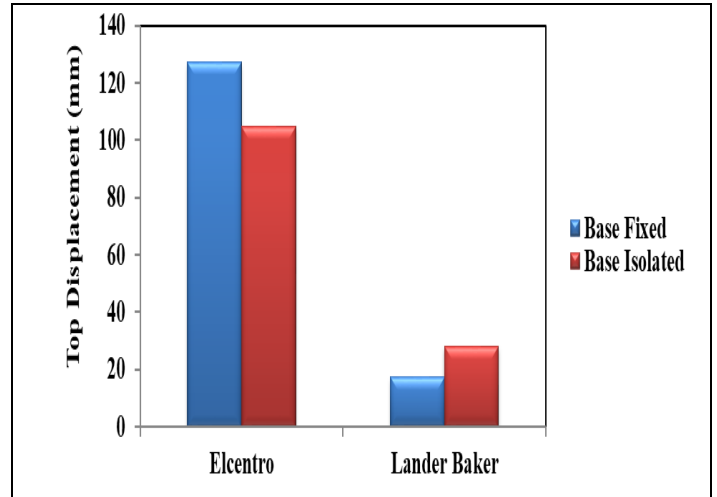


Fig. 6.8: Maximum Top Displacement

Fig. 6.8 illustrates the top displacement of the base fixed chimney and base isolated chimney under Elcentro nearfield earthquake and Landers baker farfield earthquake. This bar chart shows that top displacement under Elcentro earthquake reduces from 127.2 mm in base fixed chimney to 104.8 mm in base isolated chimney. But it increases in case of Landers baker earthquake.

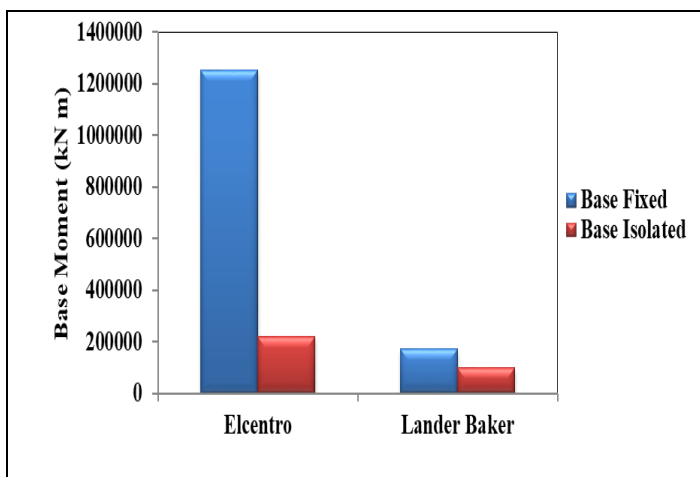


Fig.6.7: Maximum Base Moment

Fig. 6.7 is graphical representation of max. base moment of base fixed chimney and base isolated chimney under Elcentro earthquake and Landers baker earthquake. It can be seen that base moment in case of Elcentro earthquake reduces from 1251000 kNm in base fixed chimney to 220700 kNm in base isolated chimney. Under Landers baker

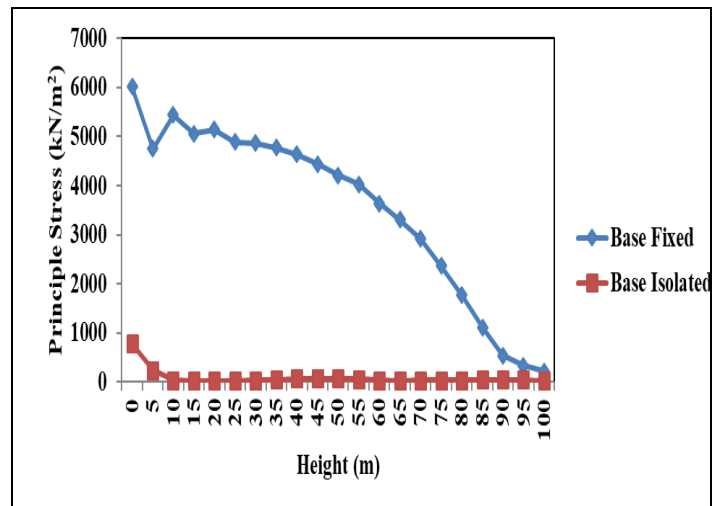


Fig. 6.9: Principle Stress v/s Height under Elcentro Nearfield Earthquake

Fig. 6.9 shows the variation of principle stresses along the height of the base fixed chimney and base isolated chimney under Elcentro nearfield earthquake. It is observed that max. principle stress is found at the base of the chimney and minimum principle stress is found at the top of the chimney.

Principle stress at the base reduces from 6016 kN/m² in base fixed chimney to 788.1 kN/m² in base isolated chimney.

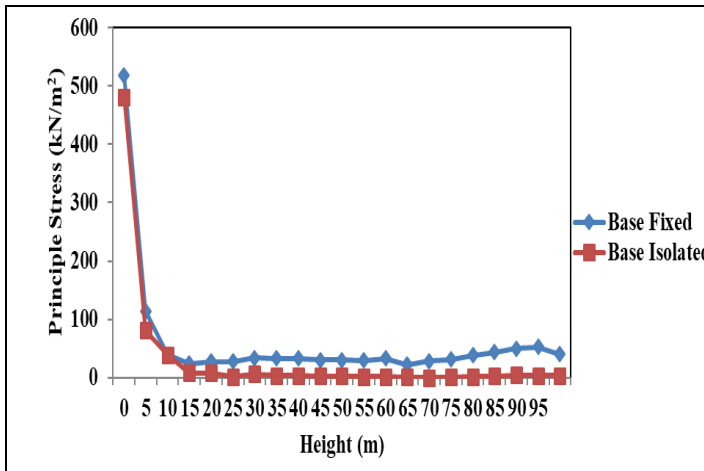
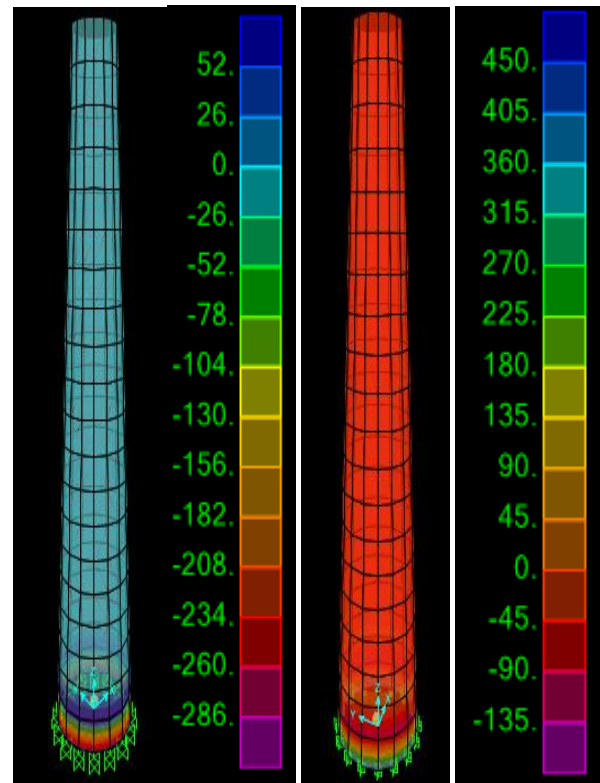


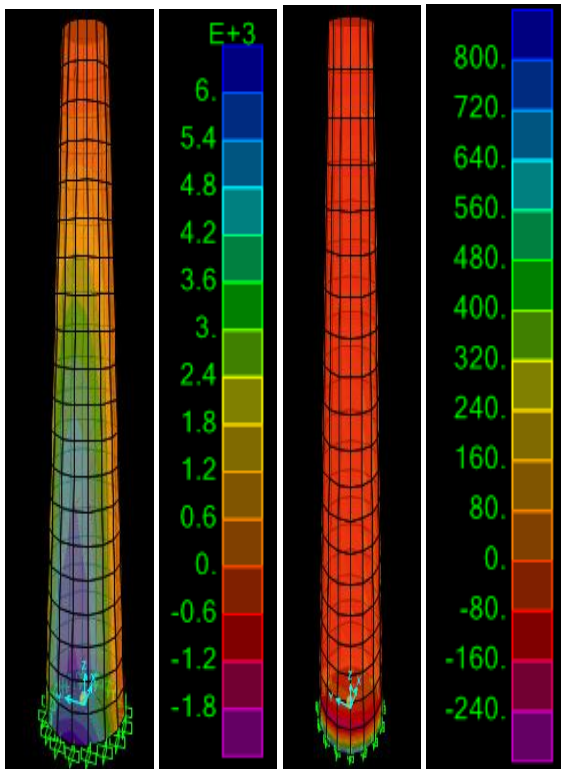
Fig. 6.10: Principle Stress v/s Height under Landers Baker Farfield Earthquake

Fig. 6.10 shows the variation of principle stresses along the height of the base fixed chimney and base isolated chimney under Landers baker earthquake ground motion. It is observed that principle stress is found at the base of the chimney and it reduces from 518 kN/m² in base fixed chimney to 481.3 kN/m² in base isolated chimney.



Base Fixed Base Isolated

Fig. 6.12: Contour Diagram of Principle Stresses under Landers Baker Farfield Earthquake



Base Fixed Base Isolated

Fig. 6.11: Contour diagram of Principle Stresses under Elcentro Nearfield Earthquake

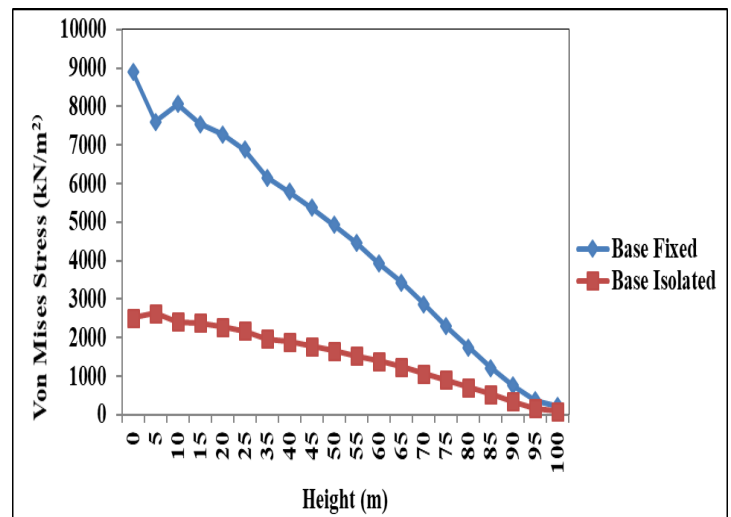


Fig. 6.13: Von Mises Stress v/s Height under Elcentro Nearfield Earthquake

Fig. 6.13 shows the variation of Von Mises principle stress along the height of the base fixed chimney as well as base isolated chimney. Von Mises principle stress is found maximum at the base of the chimney and it reduces as the height increases. At the base of the chimney its value is found

2525 kN/m² in base isolated chimney and 7600 kN/m² in base fixed chimney.

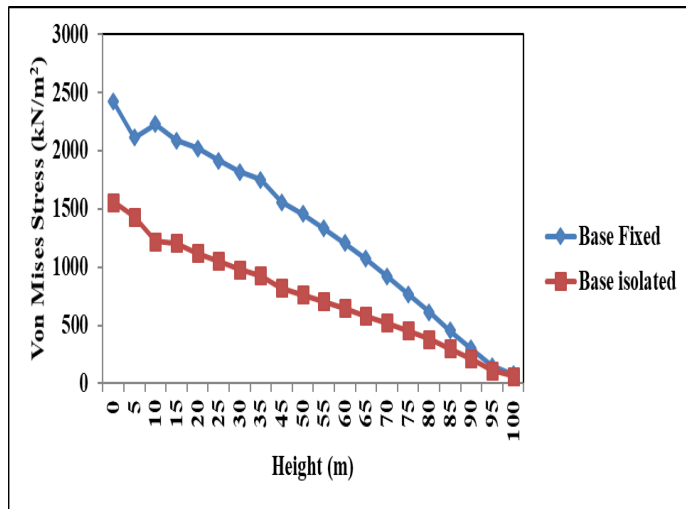
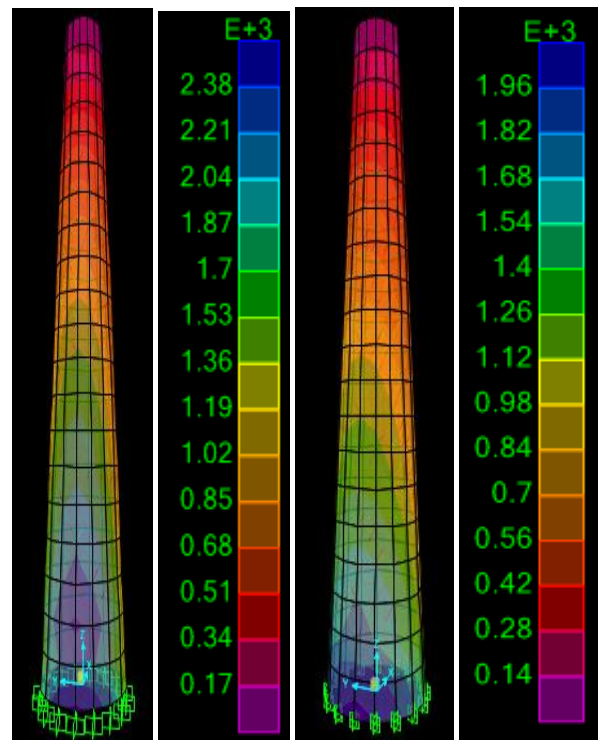


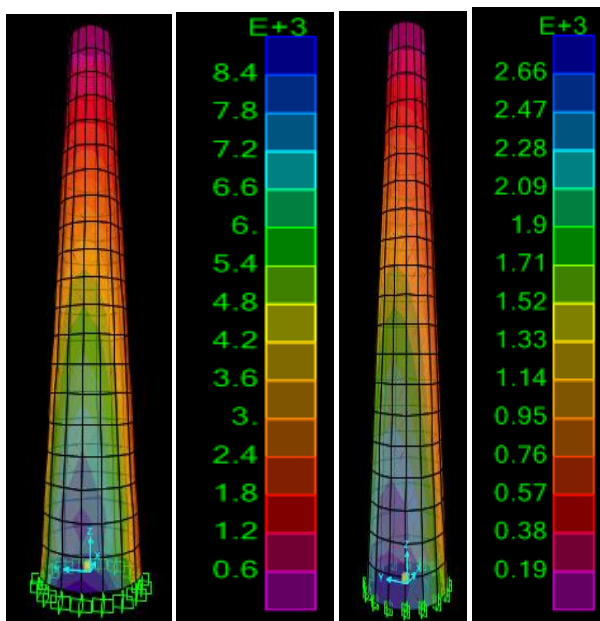
Fig. 6.14: Von Mises Stress v/s Height under Landers Baker Farfield Earthquake

Fig. 6.14 is a graphical representation of Von Mises principle stress v/s height under Landers baker earthquake ground motion. Maximum value is found at the base and minimum value at the top of the chimney. In case of base fixed chimney maximum value at the base is 2420 kN/m² and in case of base isolated chimney its maximum value is 1557 kN/m². This graph also shows that Von Mises principle stresses in case of isolated chimney get reduced.



Base Fixed Base Isolated

Fig. 6.16 :Contour Diagram of Von Mises Stresses under Landers Baker Farfield Earthquake



Base Fixed Base Isolated

Fig. 6.15:Contour Diagram of Von Mises Stresses under Elcentro Nearfield Earthquake

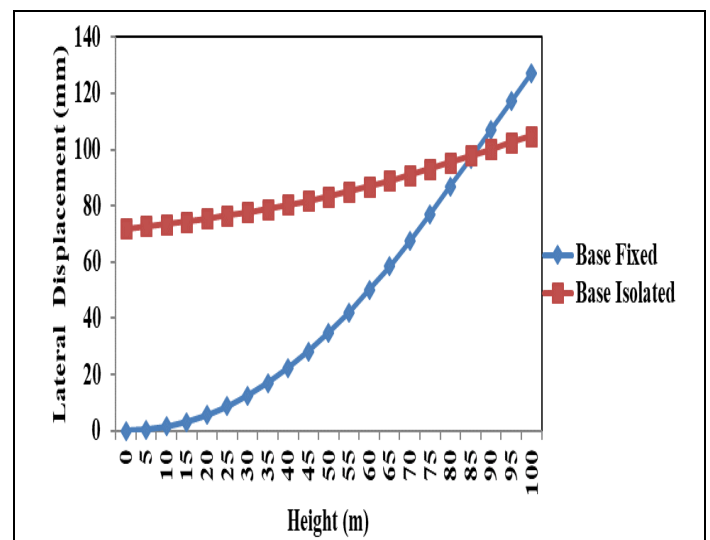


Fig. 6.17: Lateral Displacement v/s Height under Elcentro Nearfield Earthquake

Fig. 6.17 shows the variation of lateral displacement along the height of the chimney under Elcentro nearfield earthquake. In both the cases displacement is at the top of the chimney. It is observed that in case of base isolated chimney support also displaces by an amount of 71.95 mm. From this figure it is also observed that in case of base fixed

chimney displacement varies nonlinearly while in isolated case it varies linearly.

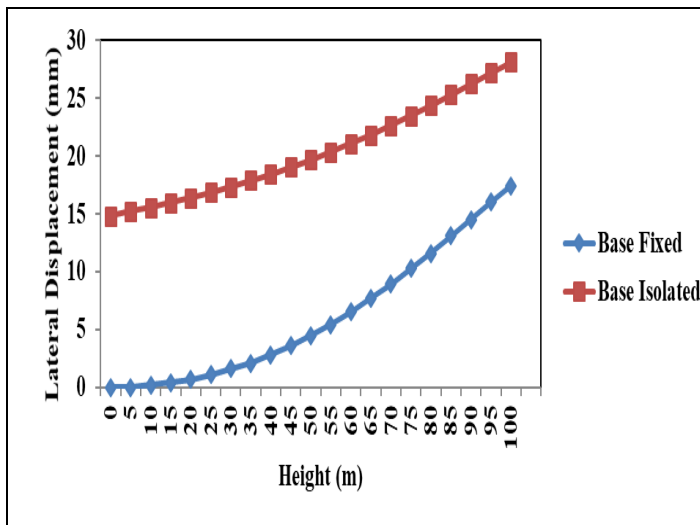


Fig. 6.18: Lateral Displacement v/s Height under Landers Baker Farfield Earthquake

Fig. 6.18 shows the variation of Lateral displacement of the chimney under Landers baker farfield earthquake along the height of the chimney. In both the cases, base fixed and base isolated maximum displacement is found at the top of the chimney. In case of isolated chimney

Base displacement is found as 14.79 mm. This figure also shows that in case of base fixed chimney displacement varies nonlinearly while in isolated case it varies linearly.

6.4 Check for Stability

Weight of the chimney, $W = 52831.95 \text{ kN}$

Radius at the base of the chimney, $R = 5 \text{ m}$

Considering maximum of Elcentro and Landers baker earthquake:

Overturning Moment, $M_o = 220700 \text{ kN-m}$

Restoring Moment, $M_R = W \times R$

$$= 52831.95 \times 5$$

$$= 264159.75 \text{ kN-m}$$

Factor of safety against overturning,

$$F.O.S. = \frac{\text{Restoring Moment}}{\text{Overturning Moment}} = \frac{264159.75}{220700}$$

$$= 1.197 > 1 \text{ (Safe against overturning)}$$

6.5 Discussion

R.C.C. chimney of circular section considering base fixed and base isolated is analysed in the present study for two different earthquake ground motions one is nearfield and other is farfield and the results obtained using SAP2000 software based on finite element method are discussed in the following paragraphs:

6.5.1 Effect of Nearfield and Farfield Earthquake

From the results it is observed that near field earthquake ground motion causes greater seismic responses in comparison with far field ground motions, because the structure in near field earthquake is located near the fault line and PGA of near field earthquake is higher than the far field earthquake.

1. Table 6.2 and Fig. 6.6 shows that base shear under nearfield earthquake ground motion is 5.79 times than that under farfield earthquake.

2. Base moment under nearfield earthquake is 7.28 times than that under farfield earthquake ground motion as shown in Fig. 6.7.

3. Fig. 6.8 and Table 6.4 shows that top displacement under nearfield earthquake is 7.32 times than that under farfield earthquake.

4. Fig. 6.9 and 6.10 illustrates that principle stress is found maximum near the base of the chimney and under nearfield earthquake it is 10.82 times than that under farfield earthquake ground motion.

6.5.2 Effect of Base Isolation

From the results it is observed that base isolation significantly reduces the seismic responses under strong ground motion, because of the decoupling of superstructure from the earthquake ground motion by introducing a flexible interface between the foundation and the base of structure.

1. Fig. 6.5 shows that Fundamental time period of the structure with base isolated is found 4.8 times than that of the base fixed structure. Effectiveness of isolation device is found considering the percentage controlled in base shear, base moment, and top displacement.

2. Table 6.2 and Fig. 6.6 shows that under nearfield earthquake base shear is controlled by 84.94% and under farfield earthquake it is reduced by 54.05%.

3. In case of nearfield earthquake ground motion percentage reduction in base moment is found 82.36% while in case of farfield earthquake it is 41.62% as shown in Fig. 6.7 and Table 6.3.

4. Table 6.4 and Fig. 6.8 shows that top displacement of chimney under nearfield earthquake is reduced by 17.64 % and under farfield earthquake no control is achieved.

5. Principle stress is found maximum near the base of the chimney and under nearfield earthquake it is reduced by 86.90% and under farfield earthquake it is reduced by 7.1% as shown in Fig. 6.9 and 6.10 respectively.

6. Von Mises principle stress is found maximum at the base of the chimney, under near field earthquake it is reduced by 71.63% while in case of far field earthquake it is reduced by 35.66%.

Conclusions:

After performing non-linear time history analysis for near field and far field earthquake considering base of the chimney as fixed and as isolated, different results are presented in comparative form and from those results following conclusions are drawn:

1. Seismic responses under nearfield earthquake are found very high as compared to farfield earthquake. So it is better to design structure those are near the fault with high period and special consideration.

2. It is better to construct the structures which are near the fault more ductile in order to reduce the seismic response.

3. Shell stress is found maximum near the base of the chimney that is why most of the chimneys fail at the base or near the base.

4. From the Figures and Tables in section 6.3 it is observed that laminated rubber bearing is quite effective in reducing the seismic responses such as base shear, base moment, shell stresses and lateral displacement of the chimney.

References

1. Veena R N, Suresh S, "Analysis and Design of R.C. Chimney", International Journal of Mechanical Engineering and Information Technology, Volume 04, Issue 01, 2016.
2. Bashar Faisal Abdul Kareem, "Thermal Analysis of Chimneys by Finite Element", Al-Mansour Journal, Issue 25, 2016.
3. Amit Nagar, M. Shiva Shankar and T. Soumya, "Non Linear Dynamic Analysis of R.C.C. Chimney", International Journal of Engineering Research and Technology, Vol. 4, Issue 7, July-2015
4. Yoganatham. C, Helen Santhi. M, "Modal Analysis of R.C.C. Chimney", International Journal in Civil Engineering, Architecture and Design Vol. 1, Issue 2, October-December, 2013.
5. Alok David John, Ajay Gairola, Eshan Ganju and Ananth Guptha "Design Wind Loads on Reinforced Concrete Chimney- An Experimental Case Study", ELSEVIER, Vol. 14, 2011.
6. Masaru Kikuchi and Ian Aiken, "An analytical hysteresis model for elastomeric seismic isolation bearings", Earthquake Engineering and Structural Dynamics, Vol. 26, Issue 2, February 1997.
7. R. S. Jangid and J. A. Kulkarni, "Rigid body response of base-isolated structures, " Journal of Structural Control", Vol. 9, 2002.
8. Necdet Torunbalci, "Seismic Isolation and Energy Dissipating Systems in Earthquake Resistant Design", 13thWorld Conference on Earthquake Engineering, Canada, August 2004.
9. Kishan Bhojani, Vishal B. Patel and Snehal V. Mevada, "Seismic Vibration Control of Building with Lead Rubber Bearing Isolator", International Conference on Research and Innovations in Science, Engineering & Technology, Kalpa Publications in Civil Engineering, Vol. 1, 2017.
10. Niraj V. Shinde and Devesh P. Soni, "Impact Response of Base Isolated Building with Adjacent Structures," International Conference on Research and Innovations in Science, Engineering & Technology", Kalpa Publications in Civil Engineering, Vol.1, 2017.
11. Pacific Earthquake Engineering Research Center, PEER (1998)^{1st} Peer Site on Characterization of Special Source Effects, University of California, Sandiego.
12. IS 1893 Part1: 2016, "Indian Standard Criteria for Earthquake Resistant Design of Structures", Sixth Revision, Bureau of Indian Standards, New Delhi.
13. IS 1893 Part2: 2015, "Indian Standard Criteria for Earthquake Resistant Design of Structures", First Revision, Bureau of Indian Standards, New Delhi.
14. S. K. Duggal, "Earthquake Resistant Design of Structure", Oxford University Press, 2007.
15. Pankaj Agrawal and Manish Shrikhande, "Earthquake Resistant Design of Structures", published by Asoke k. Ghosh, Prentice Hall of India Private Ltd., New Delhi, 2006.
16. T. K. Datta, "Seismic Analysis of Structures", John Wiley & Sons (Asia) Pte Ltd

BIOGRAPY



Alqama Hasan, Student of M.Tech-Structure and Foundation Engg. (Al-Falah University)



Gyan Singh, Structure Consultant (YG Consulting & Engineers LLP) and Working as a DGM in Gulshan Homz Pvt Ltd.



Dr. Misbah Danish Sabri, Asst. Professor (Al-Falah University, Haryana)