SEISMIC RESPONSE OF EQUIPMENT AND SERVICES IN INDUSTRIAL BUILDINGS

A DISSERTATION

Submitted in partial fulfilment of the

requirements for the award of the degree

of

MASTER OF TECHNOLOGY
in
EARTHQUAKE ENGINEERING

(*With Specialization in Structural Dynamics*)

By

YOGESH BHATTARAI

DEPARTMENT OF EARTHQUAKE ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY ROORKEE
ROORKEE – 247667, INDIA

MAY, 2016

CANDIDATE'S DECLARATION

I hereby declare that the work which is being presented in this dissertation entitled, **"SEISMIC RESPONSE OF EQUIPMENT AND SERVICES IN INDUSTRIAL BUILDINGS"**, in the partial fulfilment of the requirements for the award of the degree of Master of Technology in Earthquake Engineering, with specialization in Structural Dynamics, submitted in the Department of Earthquake Engineering, Indian Institute of Technology Roorkee, Roorkee is an authentic record of my own work carried out for a period from May 2015 to May 2016 under the supervision of **Dr. Yogendra Singh**, Professor, Department of Earthquake Engineering, Indian Institute of Technology Roorkee and **Dr. Praveen Khandelwal**, AGM (PE-Civil), NTPC Limited. The matter embodied in this dissertation has not been submitted by me for the award of any other degree.

Date:

Place: Roorkee

(YOGESH BHATTARAI)

CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

Date:

Place: Roorkee

Bcew Khndelal

(YOGENDRA SINGH)
Professor
Department of Earthquake Engineering
IIT Roorkee

(PRAVEEN KHANDELWAL)
AGM (PE-Civil)
NTPC Limited
Noida, Delhi

ACKNOWLEDGEMENT

I express a deep sense of gratitude to Dr. Yogendra Singh Professor and Head of Department of Earthquake Engineering, Indian Institute of Technology, Roorkee, for suggesting me the problem and for his intuitive and careful guidance, timely suggestions, inspiration and constant encouragement throughout the course of this Dissertation work. His excellent cooperation and inspiration made this Dissertation report complete by providing stimulating and profitable discussions.

I would also like to thank Dr. Praveen Khandelwal from National Thermal Power Corporation (NTPC) without whom the journey to understand industrial building would be impossible. Praveen sir gave a lot a time to make sure that i understand the building. He spends hours of time just to explain the structure .I am indeed debited to Praveen sir and thank him from the bottom of my heart.

I would also sincerely thank all of my fellow friends and the staffs of Department of Earthquake Engineering Department IIT Roorkee, who have helped me in every possible ways not only in the successful completion of my dissertation but also in making the time spent in IIT Roorkee a memorable one.

Finally, I have no adequate words to show my deep sense of gratitude for my beloved parents and my brother. Their motivation and inspiration have made me to reach this position.

(YOGESH BHATTARAI)

ABSTRACT

In Industries, the cost of equipment is near about or greater than the cost of building itself. Hence earthquake resistant or safe design of equipment is as important as the safe seismic design of building. Ground response spectrum is used as the governing hazard parameter for seismic safety of building, similarly floor response spectrum is used for safety evaluation of equipment and their connection with floor. In case of services, the inter-storey drift is the key design parameter for seismic safety.

Thermal power plants are unique and quite different structures than the normal buildings. Further, these contain drift sensitive services like heavy pipe lines, cladding, etc. and acceleration sensitive equipment and machines like low pressure and high pressure heaters, Turbo Generator, Deaerator, etc. which are critical during earthquake as damage to the these elements causes a significant loss.

Variation of peak floor acceleration (PFA) along the height serves as an important parameter to measure the earthquake response of the structure for the design of secondary systems. Different codes like Eurocode 8, Indian draft code, SIA 261 consider the variation of maximum peak floor acceleration along the height as linear, which is valid only if the first mode of the structure is considered. Participation of higher modes can significantly change the variation of PFA with height.

The floor response (de-coupled) approach is valid only in case of small equipment. In case of heavy machinery having significant weight in comparison with the building, the floor response approach does not provide a realistic estimation of the forces acting on the machines. However, these machines are generally rigid and their frequencies of vibration can be considered sufficiently different from the building frequencies of vibration. Hence a special form of coupled analysis is performed in the thermal power plants where the mass of the heavy equipment is included in the structure, and the response of the nodes at connections with equipment is used to estimate the forces in connections.

In this project different codes provision for finding the forces in the secondary system is reviewed and some suggestions are made for the improvement of the existing code. A real under construction thermal power plant's main power house building of NTPC Gadarwara is modelled in SAP 2000 software. Structual elements like beams, bracing, girder, slab, columns are modelled using proper finite elements

tools (i.e. beam element, shell element) . Connection details like moment connection, shear connection are modelled as rigid and pinned connection.

Preliminary analysis like modal analysis, p-delta analysis are done to check structure for any defects

A site specific response spectrum is prepared for the given site by probability seismic hazard assessment by IIT Roorkee and for the same site specific response spectrum a set of seven spectrum compatible time history is generated and applied at the base of the structure.

Finally the response of the deaerator is found out using the time history generated from the response spectrum. As the mode shape of the industrial building are complex and includes significant role of torsion also so to see the response of the structure the variation of Peak floor acceleration (PFA) along different column is studied. The effect of variation of response from the time histories generated from a single response spectrum is also studied.

TABLE OF CONTENT

LIST OF FIGURES

LIST OF TABLES

1 INTRODUCTION

In thermal power plant the machines like Turbo Generator, De-Aerator, LP and HP Heater plays an important role for the production of electricity. The cost of the equipment is very expensive and any failure of the equipment leads to tremendous loss of basic facility during earthquake. Hence for this reason a foundation for the calculation of the forces in the link of machines with the main structure is required.

Different codes have given the variation of the acceleration with height but they don’t give same value even though the fundamental theory governing them all is same. Nowadays for getting the exact forces a concept called Floor Response Spectra is used. This concept helps to give the exact amplification factor which is missing in codal provision.

Depending upon the mode shapes the maximum acceleration along the height varies. In case of non-linear analysis this value gets reduced due to formation of hinges and dissipation of energy.

So in this dissertation an existing ongoing model (i.e. NTPC Gadarwara) is modelled in SAP 2000 and various parameters like variation of maximum acceleration along height, amplification factors have been studied for linear case.

1.1 Structural configuration of a thermal power plant

Coals are widely used materials for the generation of electricity in India. Coal generates 59% of electricity in India [1]. NTPC Gadarwara works with coal as the basic fuel. As in industrial structures many machine are required and as machine are the heart of the industries, so its safety during an earthquake is a must issue. Figure 1.1 clearly express the working process in coal based thermal power plant.

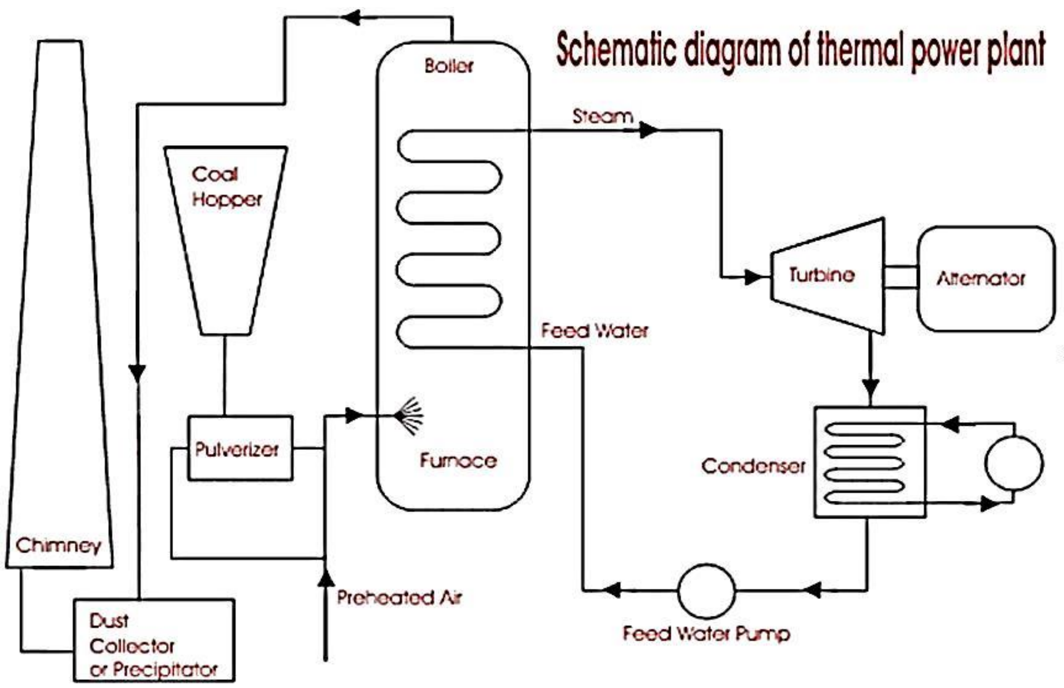


Figure 1.1 Working of thermal power plant [2]

The rotation of turbine produces electricity in the alternator. To rotate turbine high pressure and high temperature gas is required, which is produced by the boiler, which is being heated by the mixture of preheated air and pulverized coal. To make this a continuous process the outlet in turbine which is low temperature and low pressure needs to be made high temperature and high pressure. For this the outlet from turbine is send to cooling tower so the output material becomes water which is then compressed using compressor and then send to boiler where it is heated. The by-products from boiler are send to chimney via ESP which filters the harmful air.

1.2 Major machines and equipment

a) Turbo Generator

A turbo generator (Figure 1.3) is the combination of a turbine and electric generator for the production of electric power. It is the most important and expensive equipment of the power house building. It also consists of auxiliaries like condenser, pipelines carrying superheated steam. There are two Turbo Generator of capacity (800MW Each) in Gadarwara.

b) LP and HP Heater

LP heater is located between the condensate pump and either the boiler feed pump. It normally extracts steam from the low pressure turbine.

HP heater is located on the downstream of the boiler feed pump and sends the air to the Turbo Generator for running the turbine

c) Deaerator

A deaerator (Figure 1.2) is a device that removes the oxygen and other dissolved gases from the feed water to steam-generating boilers which if not removed causes corrosion on the boiler and the generator.

Figure 1.2 De-aerator [3]

Figure 1.3 Turbo-Generator [4]

1.3 Drift sensitive and acceleration sensitive elements

Acceleration sensitive element are those element which are more compact items for which relative movements between the points of support to the structure are likely to be small but it damages due to acceleration imposed by the structure on them. Usually the damage takes form of item becoming detached from support .Design strategy is to make the anchorage of the items strong to develop the shear and overturning force to prevent failure.

ASCE/SEI 31-03 are useful source of qualitative design and assessment information. some of the example of acceleration sensitive element are De-aerator, LP & HP Heater .

Displacement sensitive element are those elements that damage due to distortion on them by the structure.eg cladding element attached to the façade, Pipelines. Design strategies is to make structure stiff so that the imposed displacement are small not to cause damage(inter-storey drift helps to measure this parameter) or make items flexible to accommodate the imposed deflection, either by flexibility within the item or at the point of attachment.

1.4 Floor response spectrum

Let ω and ω_0 denotes natural frequency of primary and secondary system respectively and ξ , ξ_0 denotes the damping ratio of primary and secondary system,as shown in Figure 1.4 then for the given earthquake excitation $U_g^{**}(t)$ the basics equation of motion for SDOF for primary system is given by Equation 1.

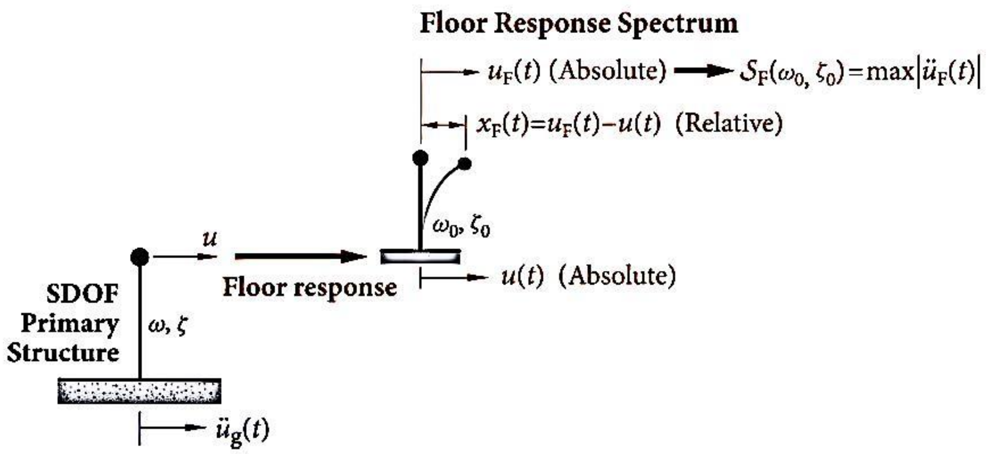


Figure 1.4 SDOF system for finding Floor response spectrum [5]

$$X^{**}(t) + 2\xi\omega X^{*}(t) + \omega^2 x(t) = -U_g^{**}(t) \tag{1}$$

$$U^{**}(t) = X^{**}(t) + U_g^{**}(t) = -2\xi\omega X^{*}(t) - \omega^2 x(t) \tag{2}$$

Where $X^{**}(t)$ and $U^{**}(t)$ represents relative and absolute acceleration of the primary system. Now consider a secondary system mounted on primary system, and then the equation of secondary system mounted in primary system is given by Equation 3 and 4.

$$X_f^{**}(t) + 2\xi_0\omega_0 X_f^{\ *}(t) + \omega^2 x_f = -U^{**}(t) \tag{3}$$

$$U_f^{**}(t) = X_f^{**} + U^{**}(t) = -2\xi_0\omega_0 X_f^{\ *}(t) - \omega^2 x_f(t) \tag{4}$$

Where $X_f^{**}(t)$ and $U_f^{**}(t)$ represents the relative and absolute acceleration of secondary system.

The maximum absolute acceleration of the secondary system $S_f(\omega_0, \xi_0) = \max|U_f^{**}(t)|$ is the floor response spectrum (FRS) for the SDOF system [5].

1.5 Past studies

There are various methods for the generation of floor response spectrum. Some of them are developed under decoupled form of analysis whose theory is based on the simple assumption to ignore the interaction effect between primary and secondary system.

In 1975, Duff [6] developed a simplified graphical method to generate FRS from given ground response spectrum. But the demerit of this method was that the pseudo ground damping is determined empirically and hence the amplification factor which depends on pseudo ground damping used gave very unrealistic results.

In 1975, Singh [7] based on random vibration theory, proposed a direct method for the calculation of FRS. But this method is not applicable when resonance of secondary system with multiple structural mode occurs .

In 1993 Yasui et al. [8] derived approximate analytical result in time domain instead of frequency domain

In 2013 An et al. [9] evaluate analytically the convolution relation between two unit impulse response function .However as they used CQC and SRSS rules for modal combination. This leads to large error in estimating FRS particularly in tuning case and when the primary system has closely spaced modes.

In 1983, Der Kiureghian et al. [10] devised perturbation technique (method to find and minimize error in every successive iteration) considering a combined primary secondary system for modal analysis for generation of FRS.

Various methods (An et al., 2013; Der Kiureghian et al., 1983; Igusa and Der Kiureghian, 1985; Singh,1980; Yasui et al., 1993) have been applied to obtain an approximate value of the tuning term . These methods give conservative results in some frequency ranges but un-conservative results in other frequency ranges due to the various approximations used. Furthermore, it is unknown when and by how much the FRS obtained is conservative or un-conservative.

In 2015, Bo li et al. [5] devised a concept of tRS(tuning response spectrum) to deal with the tuning of the equipment along with statical approaches for the estimation of tRS for a given ground response spectrum to calculate FRS from GRS directly.

In 2012, Fajfar et al. [11]showed that inelastic behaviour of the primary system reduces the peak value of the floor response.(i.e for elasto-plastic model, peak value occur in tuning region, but in case of strain hardening model peak value occurrence is shifted towards higher periods than the tuning period).

In 2011, Benno Hoffmeister et al. [12]showed that the fundamental period and the energy dissipating behaviour of the supporting structure have a significant influence on floor response spectra as dissipating behaviour reduces the floor spectra. Floor response spectra for long period structure having low spectral ground acceleration are lower than those structures having short period.

1.6 Introduction of the problem

Due to heavy weight of secondary system the weight of secondary system cannot be ignored in the analysis. This type of analysis comes under special form of coupled analysis. Here it is assumed that the stiffness of the secondary system is very high such that the floor response itself becomes the response of the secondary system. Codal provision for finding floor response is given taking in consideration of regular residential building but the industrial structure is a complex building and dominant role of torsion makes many simplified assumption like variation of max acceleration along the height being linear, principle of mode superposition etc. becomes invalid. A check of all this principle with an existing model can only make the validity of the existing theory regarding the generation of coupled floor response.

1.7 Objective of this dissertation

The primary objective of this dissertation is to model a real under construction power house building by NTPC which is located in Gadarwara Town of Narsinpur District of Madhya Pradesh and then find out floor response for important floors carrying machines like LP Heater, HP Heater and Deaerator. The specific objectives of the dissertation are:

i) To study various methods for estimating response of equipment in buildings.

ii) To review different codes for provisions on amplification of floor acceleration along height of the building.

iii) To make a numerical model of NTPC Gadarwara main power house building and to find out floor response spectrum.

iv) To study the variation of PFA along the height of the considered thermal power plant.

1.8 Organization of this dissertation

(i) Chapter 1 gives the introduction of floor response spectrum, problems associated with the generated floor response spectrum, Drift sensitive and Acceleration sensitive element, and past studies that are done on generation of floor response.

(ii) Chapter 2 gives the different methods to generate floor response spectrum, their merits and demerits. It discuss about coupled and decoupled form of analysis, different formulas for variation of peak floor acceleration along the height given by the code, simplified method of generating FRS considering it to be SDOF etc.

(iii) Chapter 3 gives the Modelling of thermal power plant. It gives information of site taken for the project, different types of connection used for joining the members, loading details.

(iv) Chapter 4 gives the analysis method for the generation of floor response like Direct integration method, Modal time history method. This chapter also describes about the procedure to generate time history from given ground response spectrum.

(v) Chapter 5 is for the results obtained from the analysis of the thermal power plant like modal analysis, variation of PFA with height, Response of the De-aerator.

(vi) Chapter 6 discusses about the conclusion drawn from the study.

2 DIFFERENT METHODS OF FINDING SEISMIC RESPONSE OF EQUIPMENT IN BUILDINGS

Different method of finding seismic response of equipment in buildings as per literature review are reviewed and described below [12]

a. Simplified methods in different codes

b. Decoupled time history analysis

c. Coupled time history analysis

d. Floor response spectrum from direct Ground response spectrum

e. Simplified approach from R. Villaverde

2.1 Simplified methods in different codes [13]

The basics concept of distribution of acceleration for different code is it varies linearly with height. Various code and formula to calculate seismic force at non-structural element level are given in Table 1.

Table 1 Comparision of Various Code for finding seismic force in secondary system

Code	Formula for calculation seismic force in non-structural element
Euro code 8	$F_a = \frac{s_a \gamma_a}{q_a} w_a$ (5) *where* $s_a = \frac{a_g}{g} s \left[\frac{3\left(1+\frac{z}{H}\right)}{1+\left(1-\frac{T_a}{T_1}\right)^2} - 0.5 \right]$ (6)
Uniform building code (UBC 1997)	$F_p = 4C_a I_p w_p$ (7) $F_p = \frac{a_p C_a I_p}{R_p}\left(1+\frac{3h_x}{h_r}\right) W_p$ (8) *where* $0.7C_a I_p W_p \le F_p \le 4C_a I_p W_p$ (9)

International Building code (IBC 2003)	$F_p = \frac{\left(0.4 a_p S_{DS} W_p\right)}{\frac{R_p}{I_p}}\left(1+\frac{2z}{h}\right)$ *(10)* *where* $0.3 S_{DS} I_p W_p \le F_p \le 1.6 S_{DS} I_p W_p$ *(11)*
New Zealand code (NZS 4203:1992)	$F_{ph} = C_{ph} W_p R_p$ *(12)* $F_{pv} = C_{pv} W_p R_p$ *(13)*
Draft code Is 1893	$F_p = \frac{Z}{2}\left(1+\frac{x}{h}\right)\frac{a_p}{R_p} I_p W_p \ge 0.1 W_p$ *(14)*
SIA 261	$F_P = \frac{a_{gd} S}{g} \frac{2\left(1+\frac{Z_a}{h}\right)}{1+\left(1-\frac{T_a}{T_1}\right)^2} \frac{\gamma_F G_a}{q_a}$ *(15)*
ASCE-7-05	$F_p = \frac{0.4 a_p S_{DS} W_p}{\left(\frac{R_p}{I_p}\right)}\left(1+2\frac{z}{h}\right)$ *(16)* *Where 0.3* $S_{DS}\ I_p\ W_p \le F_p \le 1.6\ S_{DS}\ I_p\ W_p$ (17)

Where

γ_a = importance factor.

w_a = weight of the element.

a_g = design ground acceleration.

g = acceleration of gravity.

T_a = fundamental period of the non-structural element.

T = fundamental period of the building in the relevant direction.

Z = height of the non-structural element above the base of the building.

H= total height of the building.

S= soil factor.

q_a =behaviour factor for non-structural elements.

C_a = Horizontal seismic coefficient.

a_p = Component amplification factor.

I_p = Importance factor of the non-structural element

R_p = Component response modification factor.

W_a= Weight of the element.

S_{DS} =spectral acceleration at short period

R_p = component response modification factor.

I_p= importance factor of the component.

z = height of point of attachment of component with respect to the base.

h= average roof height of the structure with respect to the base

W_p = weight of the component

a_p = component amplification factor.

Z= Zone factor .

x= Height of point of attachment of the non-structural element above top of the foundation of the building

h= Height of the building

a_p=Component amplification factor.

R_p= Component response modification factor.

I_p= Importance factor of the non-structural element.

W_p= Weight of the non-structural element.

Eurocode 8 [14]

It takes into account of ground motion, soil factor, structural amplification, and importance of non-structural element. This code clearly states that very important non-structural element to be analysed by making a realistic model of the structure using floor response spectrum. For other elements that may cause risks to persons, or affect the main structures or services of critical facilities be verified to resist design seismic load F_a given by equation (5) and (6).

Uniform building code (UBC1997) [15]

Here equation (iii) gives conservative result as it consider the non-structural element is subjected to four times the peak ground acceleration without consideration to the location of element in building, whereas equation (8) is more accurate and reliable as it consider the different factor on which response of non-structural element depends. Equation (9) gives upper and lower bounds for the force by considering element rigid and flexible.

International building code (IBC2003) [16]

Equation (10) computes design seismic force assuming input acceleration at the ground floor equal to the peak ground acceleration (0.4SDS) and that at the roof of the building is equal to three times the peak ground acceleration. Equation (11) shows upper and lower bound for the calculated design seismic force.

New Zealand code (NZS 4203:1992) [17]

Equation (12) and equation (13) gives horizontal and vertical seismic force calculation.

Draft code IS 1893 [18]

It assumes the linear variation of acceleration of 0.5Z at ground and twice the value of acceleration at roof than the ground.

Indian standard IS 1893 Part1 (2002) [19]

There is no formula developed for calculation of forces in the non-structural element. However it clearly states that in important cases it is recommended to obtain floor response spectra.

SIA 261:2003 [20]

SIA code is similar to Eurocode as it considers the period of the secondary system. A direct comparison of Eurocode and SIA code is possible and is shown in and Figure 2.1

ASCE-7 [21]

This code gives the maximum and minimum limit for the seismic force considering structure to be rigid and flexible, respectively.

Table 2 PFA variation along height for different normalised period (T_a/T_1) in Eurocode

		T_a/T_1								
		0	0.5	1	1.5	2	2.5	3	3.5	4
z/h	0	1	1.9	2.5	1.9	1	0.42	0.1	-.09	-0.2
	0.25	1.38	2.5	3.25	2.5	1.38	0.65	0.3	0.02	-.13
	0.5	1.75	3.1	4	3.1	1.75	0.88	0.4	0.12	-0.1
	0.75	2.13	3.7	4.75	3.7	2.13	1.12	0.6	0.22	0.03
	1	2.5	4.3	5.5	4.3	2.5	1.35	0.7	0.33	0.1

Table 3 PFA variation along height for different normalised period (T_a/T_1) in SIA261

		T_a/T_1								
		0	0.5	1	1.5	2	2.5	3	3.5	4
z/h	0	1	1.6	2	1.6	1	0.62	0.4	0.28	0.2
	0.25	1.25	2	2.5	2	1.25	0.77	0.5	0.34	0.25
	0.5	1.5	2.4	3	2.4	1.5	0.92	0.6	0.41	0.3
	0.75	1.75	2.8	3.5	2.8	1.75	1.08	0.7	0.48	0.35
	1	2	3.2	4	3.2	2	1.23	0.8	0.55	0.4

In the Table 2 and Table 3 the green box in vertical direction represents different z/h ratio and the green box in horizontal direction represents different time period ratio.

Here clearly we see negative value in Table 2 for higher time period ratio. This implies that for highly flexible secondary system Eurocode 8 gives negative acceleration value which has no physical interpretation. This negative value can be avoided by introducing minimum force limit like in case of IBC 2003 and UBC 1997.

Moreover as per Table 2 and Table 3 and with reference to Figure 2.1 the value of acceleration in SIA 261 are smaller than the value as given by Eurocode 8 for T_a/T_1 between (0-2.5) and after that time period ratio the opposite is true.

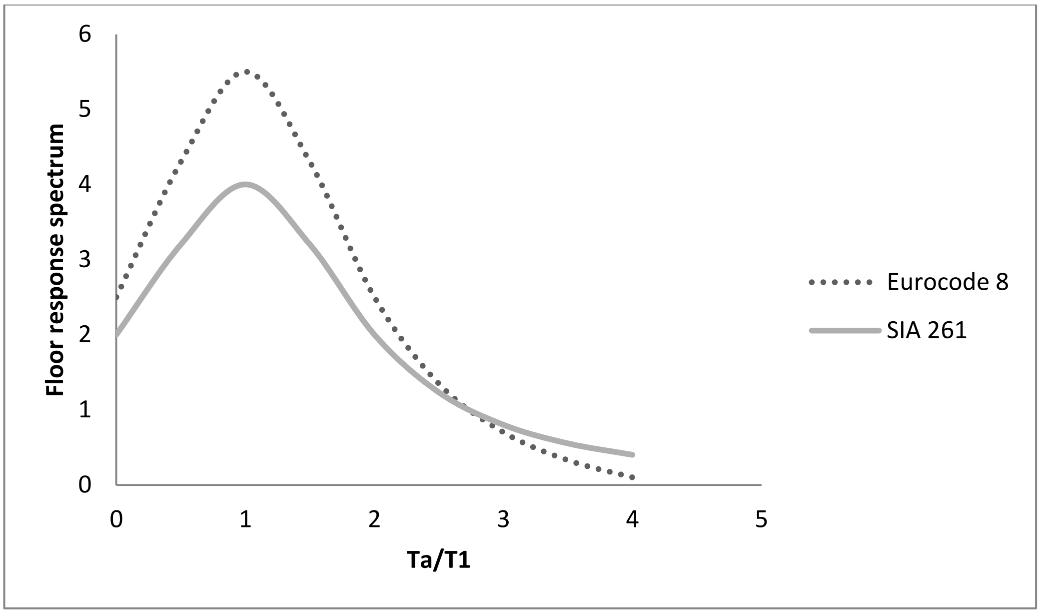


Figure 2.1 Floor response spectrum at the top of building (Eurocode 8 and SIA261)

A comparison of peak floor acceleration between all the codes, assuming secondary structure to be rigid and subjected to unit PGA, is given in Table 4.

Table 4 Comparison of Variation of PFA along height for rigid SS

Z/H	Eurocode	UBC	IBC	IS DRAFT	SIA	ASCE-7
0	1	1	1	1	1	1
0.1	1.15	1.3	1.2	1.1	1.1	1.2
0.2	1.3	1.6	1.4	1.2	1.2	1.4
0.3	1.45	1.9	1.6	1.3	1.3	1.6
0.4	1.6	2.2	1.8	1.4	1.4	1.8
0.5	1.75	2.5	2	1.5	1.5	2
0.6	1.9	2.8	2.2	1.6	1.6	2.2
0.7	2.05	3.1	2.4	1.7	1.7	2.4
0.8	2.2	3.4	2.6	1.8	1.8	2.6
0.9	2.35	3.7	2.8	1.9	1.9	2.8
1	2.5	4	3	2	2	3

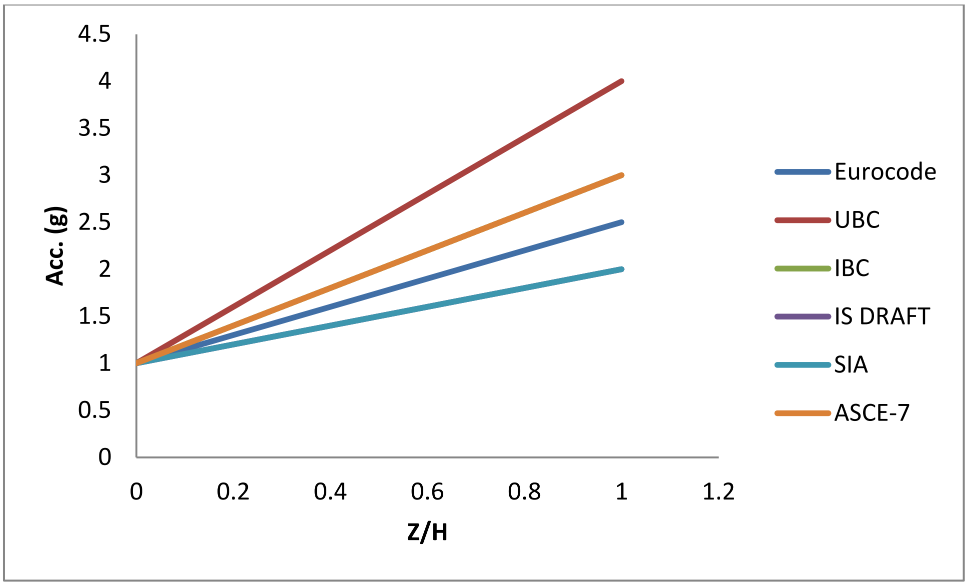


Figure 2.2 Variation of PFA along height according to different codes

Figure 2.2 shows that for a rigid secondary system the variation of acceleration along the height is linear. IS draft code and SIA have same variation pattern (i.e. PFA at the roof is 2.5 times the PGA) and similarly IBC and ASCE have same variation pattern (i.e. PFA at the roof is 3 times the PGA) .

Codal methods of finding floor response are based on some assumption which itself contains flaws some of which are described below:

i) According to different codes the seismic force on non-structural components is governed by first fundamental period and linear mode shape. This is an invalid assumption for non-regular structures as they have torsional effect as well as the participation of higher modes.

ii) The inelastic response of Primary and secondary system is neglected .The non-linearity in structure significantly reduces the floor response spectra

iii) Clearly for higher value of T_a/T_1 Eurocode 8 results negative value. This de-merits can be overcome by introducing lower limit value as like ASCE, IBC, UBC.

iv) The response of primary structure with respect to response spectrum is not considered as only PGA and soil factor is used for finding PFA along height.

v) There is variation of amplification factor for different codes. Eurocode uses 5.5, whereas ASCE and SIA use 4.

2.2 Decoupled time history analysis

The other name for this type of analysis is Floor response spectrum method. As per IS: 1893 (Part 1) draft code, it clearly states that "For non-structural elements of great importance or of a particular dangerous nature, the seismic analysis should be based on the floor response spectra derived from the response of the main structural system."

Floor Response spectrum can be defined mainly by these parameters: [22]

(a) Spectrum value at low frequency

(b) Spectrum value at high frequencies, i.e.Peak floor acceleration; and

(c) Frequency location and magnitude of the major spectrum peak. zones.

In view of the current trend toward standardizing the design of thermal plants, the development of the standard equipment design spectra becomes very useful. Floor response for different damping is shown in Figure 2.3

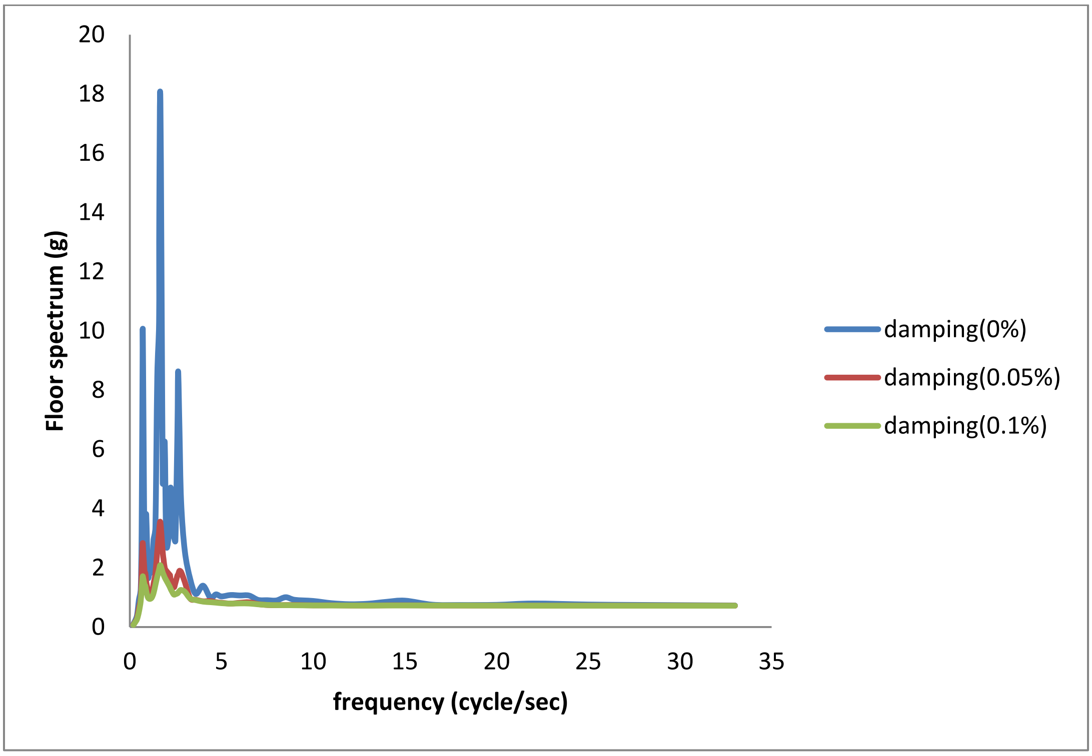


Figure 2.3 Floor response at 51m height of thermal power plant at Gadarwara

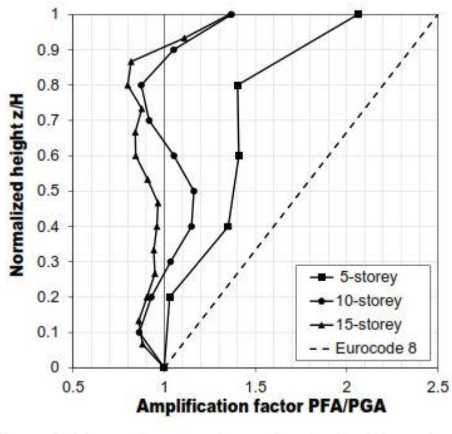
Floor response spectrum can be very well described with the help of following description and this can be used as a rough check for checking the validity of the generated floor response spectrum.

a) For Spectrum at low frequencies (Those frequencies which are less than one-third of the fundamental structure frequency) the floor acceleration is nearly equal to ground acceleration.

b) Spectrum at high frequencies converges to the Peak floor acceleration value. It is called as zero period acceleration (ZPA) of the floor. This peak floor acceleration is a function of the ground motion and the primary system structural property. From the seismic analyses from past and current industrial building, the following characteristics for the PFA were observed

(i) PFA increament or decrement with the relative height depends upon mode shape and mode participation value. It is not linear as described by code which can be seen in Figure 2.4 and Figure 2.5.

(ii) The largest value of PFA for a structure does not depend on the actual height of the structure. It usually occurs at the top of the structure. For horizontal response, it is very close to 2.5-3.



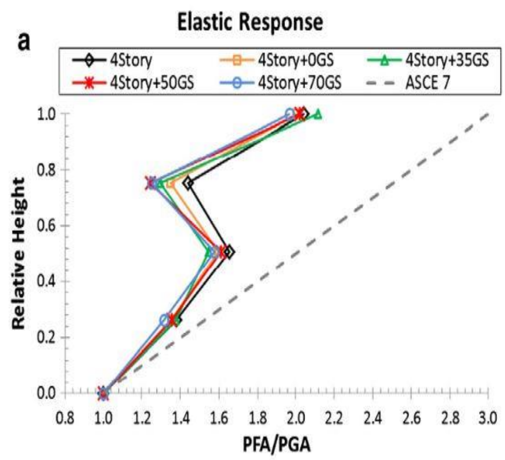


Figure 2.4 PFA variation along height [12]

Figure 2.5 PFA variation along height [23]

c) Spectrum peaks usually occurs at the fundamental frequency of the primary system and the magnitude of the spectrum peak is in inversely proportional to the damping of the primary and secondary system

2.3 Coupled time history anlaysis

Usually when the mass of equipment is very high, the decoupled form of equation results in very conservative design and gives uneconomical result. In such case both primary and secondary system are coupled in one model. This helps to give conservative results.

There are two types of coupled time history analysis which are

a. Only the mass is added to buildings and interaction between primary and secondary system is ignored

b. Secondary system mass is model as attached to primary system by spring element of certain stiffness.

Dynamic interaction between two system (primary and secondary system) can significantly reduce the acceleration response spectrum for a system having a high mass of secondary system whereas for secondary system of light weight, coupled and decoupled analysis usually shows agreement which can be seen in Figure 2.6.

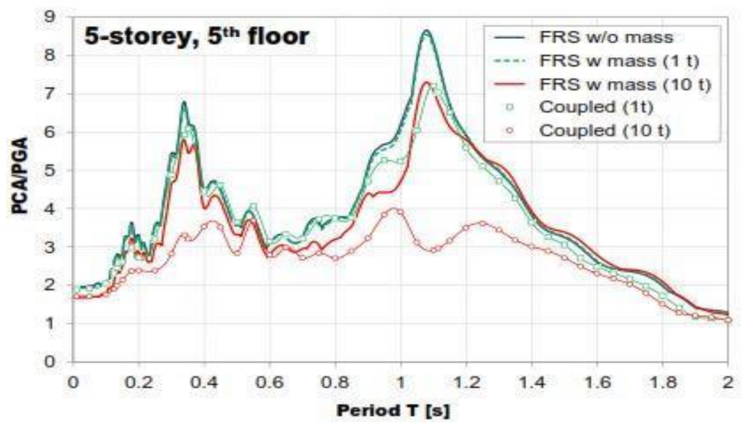


Figure 2.6 Coupled and decoupled FRS response for light and heavy weight equipment [12]

.

2.4 Coupling and decoupling criteria [24]

As per AERB guidelines for heavy reactor coupling or decoupling criteria, depends on the interaction between the building (primary system) and the equipment (secondary). The extent of interaction, depends on two mathematical parameter

a) Mass ratio (R_m)

b) Frequency ratio(R_f)

$$R_m = \frac{Total\ mass\ of\ secondary\ system}{Total\ mass\ of\ primary\ system} \tag{18}$$

$$R_f = \frac{Fundamental\ frequency\ of\ secondary\ system}{Fundamental\ frequency\ of\ primary\ system} \tag{19}$$

For a MDOF, the mass ratio is taken as modal mass ratio and the frequency ratio is taken based on the frequencies of these modes. Modes with modal mass > 20% are included.

Following are the decoupling criteria:

(i) If $R_m < 0.01$, decoupling can be done for any R_f

(ii) If $0.01 < R_m < 0.1$, decoupling can be done if $R_f < 0.8$ or $R_f > 1.25$,

Coupling should be done if $0.8 < R_f < 1.25$

(iii) If $R_m > 0.1$, and $R_f > 3.0$, (i.e. the secondary system is rigid compared to the primary system), it is sufficient to include only the mass of the subsystem.

(iv) If $R_m > 0.1$, and $R_f < 0.33$, (i.e. the secondary system is flexible compared to the primary system), decoupling can be done.

(v) If $R_m > 0.1$, and $0.33 < R_f < 3.0$, coupling is required.

The above decoupling criteria are applicable for secondary systems with single point attachment to the primary system. For Multi-supported equipment criteria of decoupling should be based on ASCE4-98, C3.1.7.3.

2.5 Floor response spectrum from ground response spectrum

Floor response can be directly calculated by using ground response spectrum. There are three analytical approach for doing so, namely [5]

i) Random vibration

ii) Perturbation Method

iii) Duhamel's integral

A very simple method for direct conversion of floor response spectra was proposed by Yasui et al. (1993) based on Duhamel's integral [11] given be Equation 20. Authors have derived an equation for a SDOF system which is valid in all period range for the case of linear behaviour of Primary and Secondary system. It was derived, using the Duhamel integral combining responses of Primary and Secondary system. The responses in terms of absolute acceleration were analysed namely:

a. Responses of the PS and SS subjected to the ground motion

b. Response of the SS subjected to the absolute acceleration of the mass of the PS. The maximum values of responses are combined using SRSS (Square Root of Sum of Squares) combination rule to obtain the equation for the floor spectrum

$$A_s = \frac{1}{\sqrt{\{1-(T_p/T_s)^2\}^2 + 4(\xi_p+\xi_s)^2 (T_p/T_s)^2}} \sqrt{\{(T_p/T_S)^2 S_e(T_p,\xi_p)\}^2 + S_e(T_s,\xi_s)^2} \quad (20)$$

This is for SDOF system, for MDOF system the acceleration value obtained is to be multiplied with mode participation factor of particular mode and corresponding scalar quantity of the particular mode in required direction. Then finally all the mode responses can be combined using SRSS approach. i.e response of kth mode in nth node is given by Equation 21

$$R_{n,k} = \phi_{n,k} \Gamma_k A_k \quad (21)$$

Where Γ_k = Mode participation factor in k mode

For a SDOF system a simple conversion of I.S.1893 zone V soft soil response spectrum to FRS for different damping is shown in Figure 2.7 and 2.8.

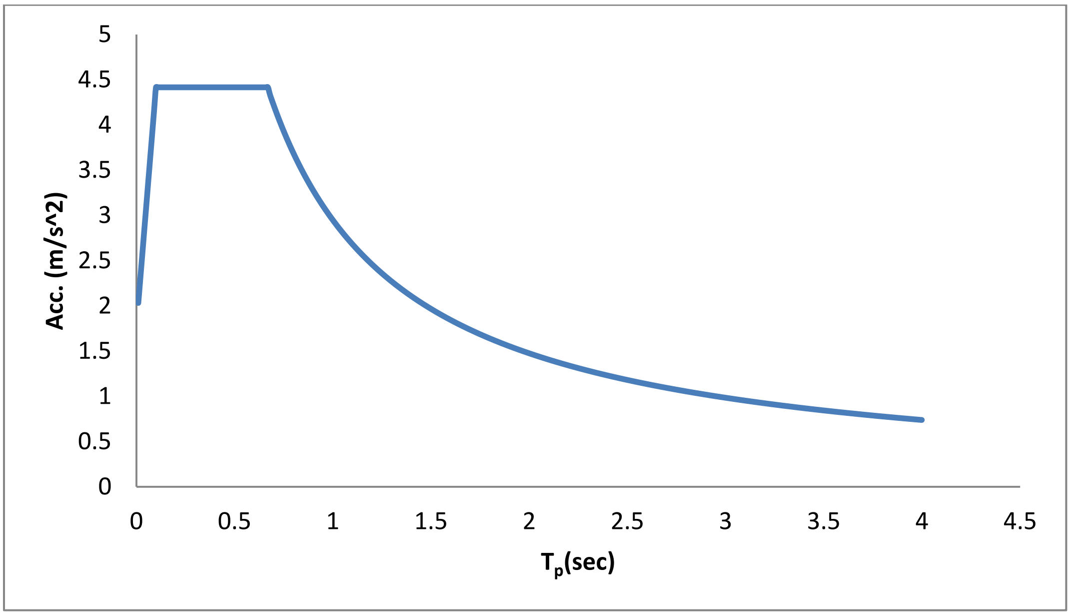


Figure 2.7 IS response spectra for soft soil zone V

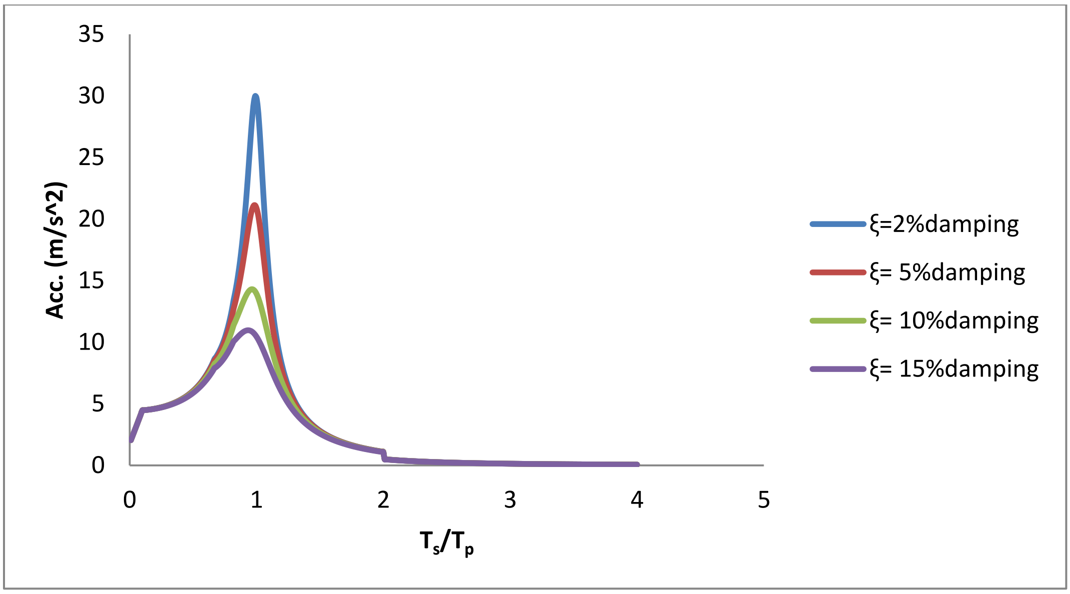


Figure 2.8 FRS for above GRS for different damping using Yasui et al.(1993) eqn.

2.6 Method by R. VillaVerde [25]

This method is based on a simplified procedure for the solving linear primary–secondary systems and then use strength reduction factors for the nonlinearity of the two subsystems. It is simple to use. Information required are geometric characteristics, weights, and target ductility of the non-structural component and its supporting structure, fundamental natural period of the structure and the response spectra for the design of the structure. Some of the basics assumption to calculate base shear in the non-structural elements are given below

a. The response of the combined structural, non-structural system is approximately given by the response in the two modes of the system that correspond to the fundamental natural periods of the two independent subsystems.

b. The fundamental natural period of the non-structural element is same as fundamental natural period of the structure.

c. The fundamental mode shape of the structure vary linearly from zero at its base to a maximum value at its top.

d. The fundamental mode of the non-structural element varies linearly along its height. In the case of a single point of attachment, it varies from zero at its point where it is connected to the structure to a maximum value at its other end and in case of two points of attachment, it varies from zero at these two attachment points to a maximum value at the point where it attains its maximum displacement

e. The generalized masses in the fundamental modes of the structure and the non-structural element are equal to their respective total masses.

f. The damping ratios in the fundamental modes of the structure and the non-structural element are equal to 5 and 0 per cent, respectively.

g. The strong part of the ground motions exciting the base of the structure exhibits a duration 25 sec.

h. Structure and non-structural component behave as independent systems for accounting nonlinear effects.

i. The strength reduction factors derived for SDOF are also valid for MDOF.

Step wise procedure of finding base shear for secondary system

i) Find ϕ_o using

$$\phi_0 = \frac{wh_{av}}{\sum w_i h_i} \tag{22}$$

ii) If time period of secondary system is not known then assuming condition of resonance find

$$c_P = \frac{1}{\sqrt{\frac{2w_P}{w} + \frac{(1+0.5T)^2 - 1}{200\phi_0^{\,2}}}} \le \frac{\sqrt{200}}{1+0.5T}\phi_0 \tag{23}$$

However if the time period of secondary system is known find c_m instead of c_p

$$C_m = \frac{\Phi_0}{\left|\left(\frac{T_P}{T}\right)^2 - 1\right|} \tag{24}$$

iii) Find base shear in the secondary system using

$$V_p = \frac{C_p}{RR_p} S_a W_p \tag{25}$$

in which S_a is the ordinate corresponding to the fundamental natural period and damping ratio of the structure in the acceleration response spectrum specified for the design of the structure, expressed as a fraction of the acceleration of gravity. However, when the fundamental natural period of the component is known, S_a represents the average of the spectral ordinates corresponding to the fundamental natural periods and damping ratios of the structure and the non-structural component. Additionally R and R_p are strength reduction factors that account for the nonlinear behaviour of the supporting structure and the non-structural component, respectively, computed using value given below. R is obtained based on the target ductility ratio for the structure and R_p based on the target ductility ratio for the non-structural component. Finally, w_p denotes the total weight of the non-structural component, and C_p is a component amplification factor.

T=time period of building,

T_p=Time period of secondary system

$$R_\mu = \begin{pmatrix} 1 + \frac{T - 0.03}{0.095}(\sqrt{2\mu - 1} - 1) \rightarrow if\, 0.03 \le T \le 0.125s \\ \sqrt{2\mu - 1} \rightarrow if\, 0.125 \langle T \langle 0.5s \\ \mu \rightarrow if T \ge 0.5s \\ \end{pmatrix} \tag{26}$$

The lateral forces for which the nonstructural component should be designed is given by

$$F_{pj} = \frac{w_{pj} l_j}{\sum_{j=1}^{n} w_{pj} l_j} V_p \tag{27}$$

F_{pj}= force acting on the j^{th} mass of the non-structural component

w_{pj}= weight of this j^{th} mass

3 MODELLING OF A THERMAL POWER PLANT

3.1 Project site and building

The industrial building in project is in construction phase (Figure 3.1) and is located near to villages Gangai, and Umaraiya (About 9 km from Gadarwara Town of Narsingpur District of Madhya Pradesh). The Feasibility report by Gadarwara STPP granted 2×800MW for stage-I phase. The mode of operation is Base load (i.e. it supplies power for 24 hours).

Figure 3.1 NTPC Gadarwara (Main power house building)

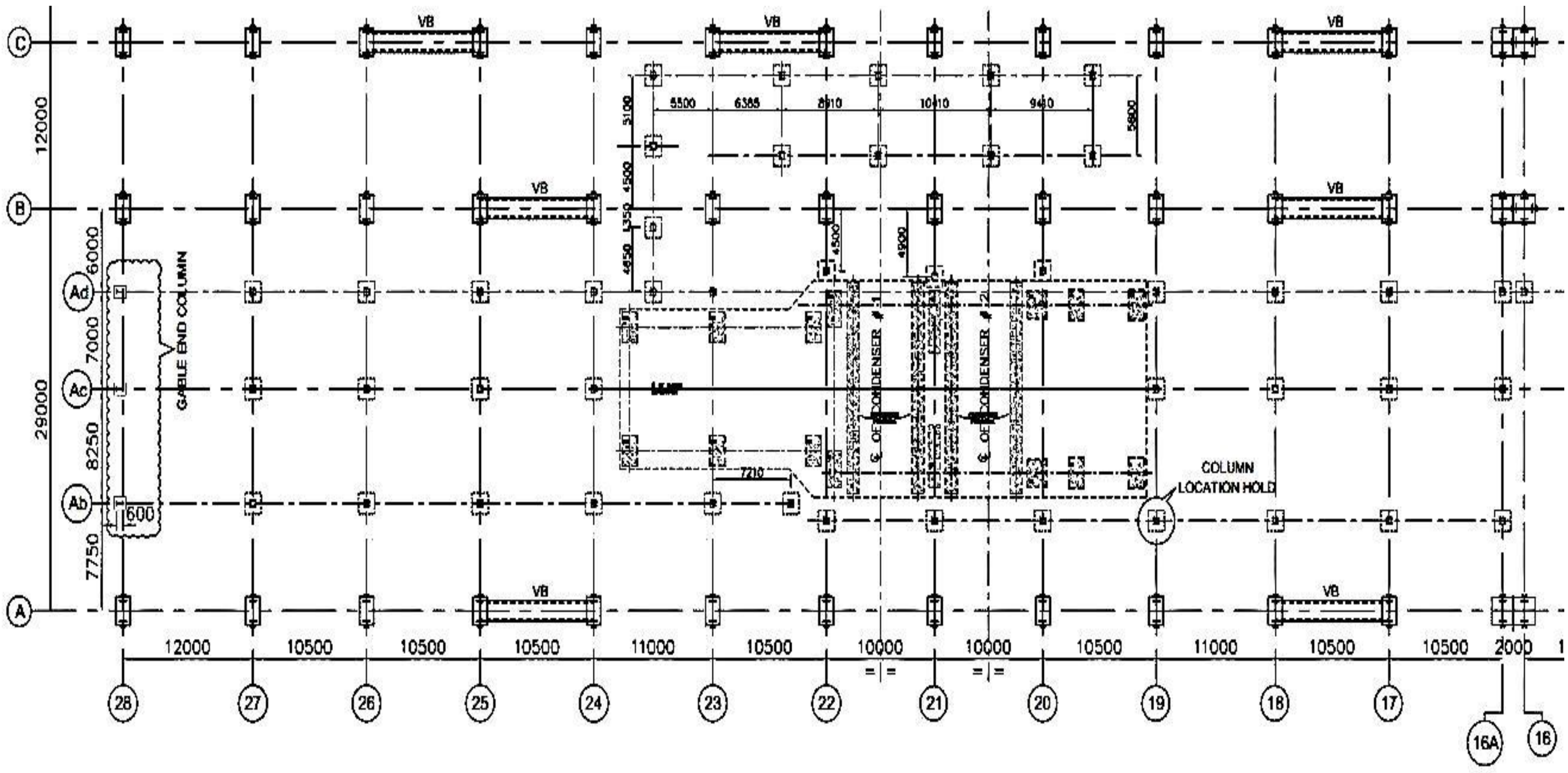


Figure 3.2 Plan at level -1.2m showing column orientation

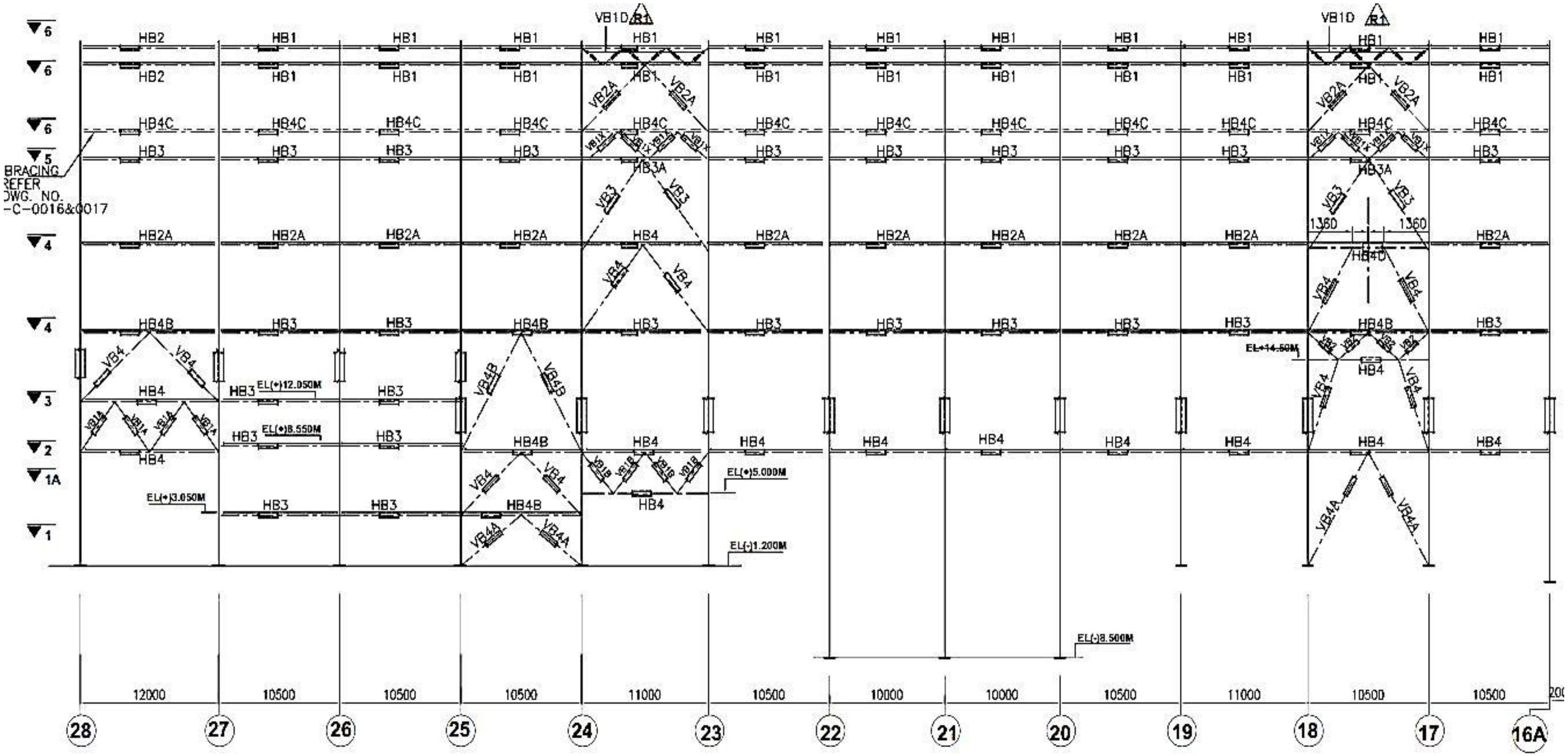


Figure 3.3 Elevation along A-Row

The plan of thermal power plant varies along the height of the structure and is designed as per the machines and equipment that are needed at that particular floor. Figure 3.3 shows the plan of NTPC Gadarwara at level -1.2 m. Here we see there are three rows along transverse direction namely A, B and C rows. This rows individual elevation is given by NTPC and they are modelled first in SAP2000 software.

3.2 Modelling of row frames

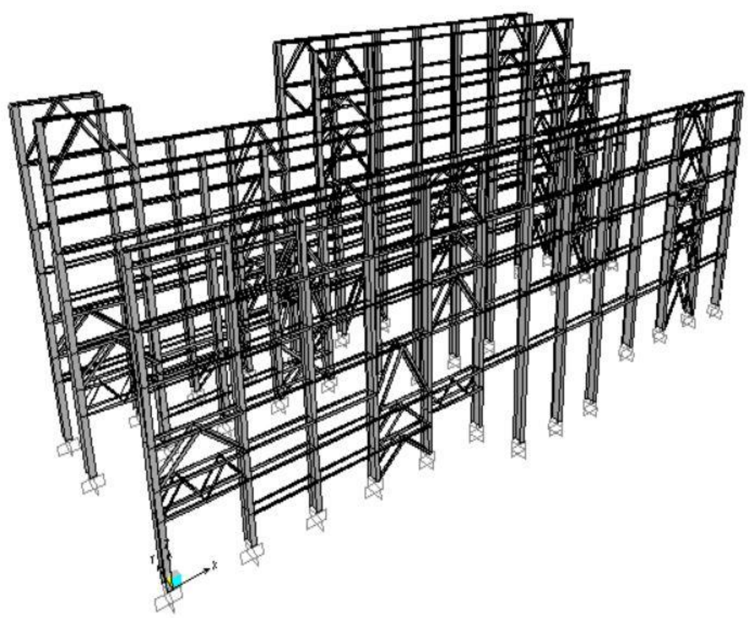


Figure 3.4 Model of A,B and C rows in SAP

3.3 Descriptions of members and joints used in row modelling

The section of columns varies with height and is built up section. Joint connection in columns is Moment connection which is model as fixed (all degree of freedom as restraint). The two columns connections in rows are done by bracing which is modelled as an axial member in SAP (i.e. M22 and M33 is free).

3.4 Modelling of beams in transverse direction of BC bay

The beams in BC bays connects two opposite columns in two rows and is found in transverse direction between B and C Rows. The joint connection of beams with the column is Moment connection.

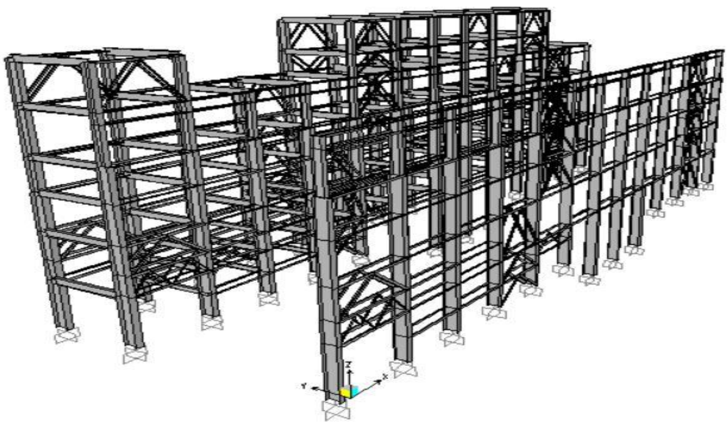


Figure 3.5 Beams in BC Bay

3.5 Modelling of roof girder

Roof girder lies between A and B row at the elevation level 40m. The connection of Girder with column in A and B row is Moment Connection. Girder is a tapered steel section with depth of 1.6m at A and B column joint and 1.31 m depth at the mid-way.

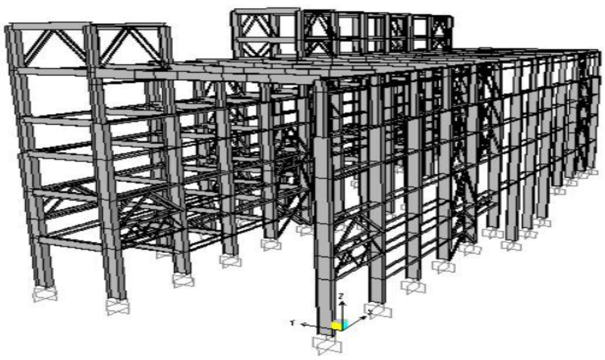


Figure 3.6 Modelling of roof girder

3.6 Modelling of auxiliary columns

Auxiliary columns are here located between A and B rows and they are called auxiliary column for the reason that they do not go up to the roof level of the building .The support foundation of all the auxiliary columns are modelled so as to give restraint to displacement in support and not the rotation.

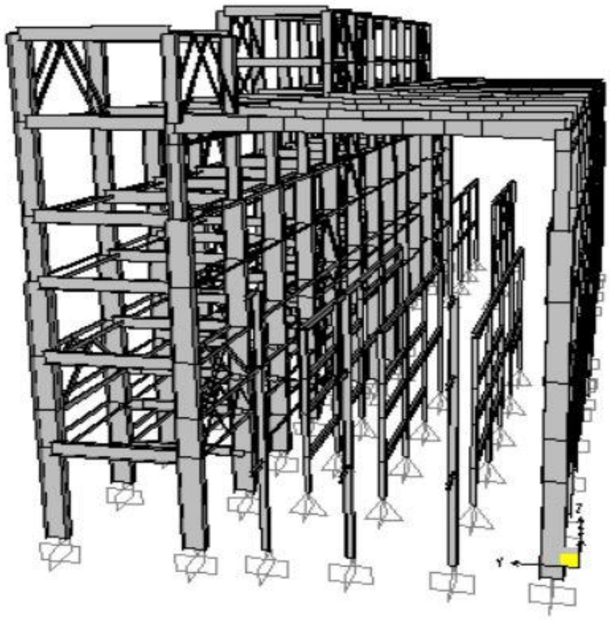


Figure 3.7 Modelling of Auxiliary columns

3.7 Modelling at different heights

There are many levels at the building each serving a unique function. Slab is modelled as shell element of 200mm thickness and zero density just to simulate stiffness of slab and weights are assigned to bays beam. Some of the levels along with its importance are given below.

Elevation at 3.5m- This Floor contains Battery.

Elevation at 5.5m- This Floor contains PTSP Cubical room. PTSP Cubicles are used for voltage metering and generator protection.

Elevation at 8.5m- It contains important equipment like LP and HP Heater. This level has a wide platform and one can easily inspect Turbo Generator from this level.

Elevation at 12.5m- It contains oil room where there are oil tank and oil cooler.

Elevation at 18m- This is similar to elevation at 8.5m, and contains Air washer unit too.

Elevation at 32m- This contains ECW Tank (Equipment cooling water system).

Elevation at 41m- This contains De-aerator tanks whose functions is to remove oxygen and other dissolved gases from water to prevent corrosion of Boiler.

Elevation at 50m- This contains framing over deaerator for piping support.

There are separate drawing for all the plan level from where the position of beam and column is obtained and model in SAP. The Final model in SAP and the on-going construction model is shown in Figure 3.8 and Figure 3.9.

Figure 3.8 Ongoing construction at site

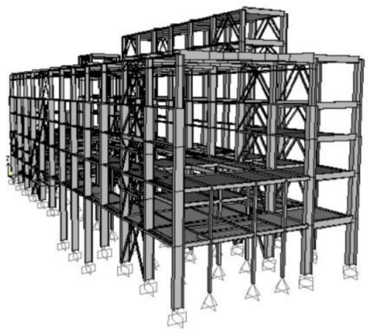


Figure 3.9 SAP model

3.8 Loading details in building

The loading details in the building are given below

1. Dead load

a) Assuming slab depth= 225mm, and concrete density=25kN/m^3, load in udl=5.625kN/m^2

b) Assuming Dead wall thickness=230mm and brick density=20kN/m^3, load in udl=4.6kN/m^2

2. Live load

For Cable vault area, Conference room, Toilet and Pantry= 10kN/m^2

For other area containing floor= 5kN/m^2, except for equipment laydown area@18m where it is assigned as=0.1kN/m^2

3. Roof live load

For Metal deck roof slab live load =1kN/m^2

For roof on BC and AB bay= 3kN/m^2

4. Piping load

Pipe load= 5kN/m^2. [Assigned as per piping drawing]

5. Equipment load

For roof bay, AHU area, CER and CCR, switchgear room= 10kN/m^2

For BMCC area, Toilet, Pantry and Conference room= 5kN/m^2

For equipment laydown area @18m= 30kN/m^2

4 ANALYSIS

The different types of analysis that are done on the structure are

a. Modal Analysis

b. Linear Modal Time history Analysis

c. Linear Direct Integration Time history Analysis

d. Non-linear Direct integration Time history analysis

4.1 Modal analysis

Modal analysis uses the overall mass and stiffness of a structure to identify the various periods at which structure will naturally resonate.

The basic equation of motion for free vibration analysis without damping is

$$[M]\{\ddot{x}\}+[K]\{x\}=0 \tag{28}$$

Assuming

$$x=a\sin\omega t \tag{29}$$

Then equation (28) can be written as

$$[K]\{\phi\}=\omega^2[M]\{\phi\} \tag{30}$$

Where

K= Stiffness matrix, ϕ= Mode shape Vector and ω= Natural frequency of the system

Modal analysis is very important in our project as it is expected to see a peak response in the floor response at the resonant time period of the primary system with the secondary system.

The Load combination that is used for seismic mass source definition is

DL+0.25LL+0.8PL+EL

4.2 P- delta analysis

P-delta is geomentric non-linearity of structure. It includes the equilibrium of structure in its deflected shape due to gravity loads (Figure 4.1) This effect leads to increase in the time period of the structure and the structure fails due to inter storey drift.

p-delta effect due to dead load need not be taken into consideration if

$$P_d \times \Delta \leq 0.25 M_p \tag{31}$$

Where, p_d = Dead load acting on column

Δ = Movement of column end in lateral direction from point of contraflexure

M_p= Plastic moment capacity based on expected strength

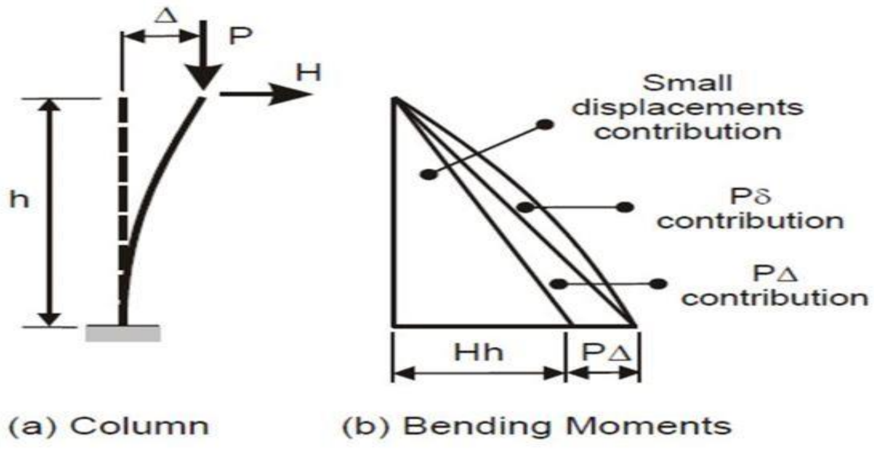


Figure 4.1 P-delta effect

Usually there are two types of p-delta effect in SAP2000 namely p-small delta and p-big delta. P-small delta is used for slender column which usually have simple curvature like in case of simple single column with heavy point load whereas p-big delta is applied for a whole multi-storeyed structure where column does not bend under single curvature.

4.3 Selection of time histories

As per AASTHO (2011) number of time history to be selected for time history to be selected is 7. The responses of the seven time history need to be average. The time history selected should be such that it should have shape close to design spectra for avoiding big scale factor.

Selection of Time history depends on many factors like type of fault, site distance from possible earthquake zone, Magnitude of earthquake etc. Filtering and baseline correction are done on raw time history for removing unwanted sound noise and correcting drift in displacement time history.

In case of project we have generated spectrum compatible time history. Response spectrum for the particular site was prepared by IIT Roorkee.

Table 5 PGA from various seismogenic sources for Gadarwara site, Madhya Pradesh

SL.No	Sources	Magnitude	R(km)	Max. Acc(g)
1	L.L	6.5	25	0.18
2	Son Narmada North Fault	7	110	0.06
3	Son Narmada south Fault	7	145	0.04
4	Tapti North Fault	6.5	214	0.01
5	Gavilgarh Fault	6.5	241	0.01
6	Tan Shear	6.5	248	0.01
7	Central Indian Shear	6.5	313	0.004
8	Great Boundary Fault	7	310	0.007

R= Horizontal distance to surface projection of the fault. (Boore and Atkinson,2008)

Equation for response Spectra normalised to 1g for various value of damping for Gadarwara power plant

$$S_a / PGA = \begin{pmatrix} 1 & 0.00 \le T \le 0.030 \\ (T/0.030)^a & 0.030 \le T \le T_1 \\ A & T_1 \le T \le T_2 \\ V/T & T_2 \le T \le T_3 \\ D/T^2 & T \ge T_3 \end{pmatrix} \quad (32)$$

Table 6 Parameters to be used in Eqn. (18) For finding GRS for Gadarwara site

Damping	α	T_1(s)	A	T_2(s)	V(s)	T_3(s)	D(s)
0.8	1.346	0.11	5.75	0.4	2.3	3.2	7.36
1	1.269	0.11	5.2	0.4	2.08	3.2	6.656
1.6	0.978	0.12	3.88	0.46	1.785	3.6	6.425
2	0.876	0.123	3.44	0.48	1.651	3.7	6.275
3	0.729	0.127	2.86	0.49	1.43	3.9	5.72
5	0.568	0.13	2.3	0.5	1.15	4	4.6
7	0.455	0.13	1.95	0.5	0.975	4	3.9
10	0.366	0.13	1.71	0.5	0.855	4	3.42

So as per the Table 5 the highest PGA of all the earthquake (i.e. 0.18g) is taken and response spectrum for 2% damping is constructed using Table 6 for generating spectrum compatible time history.

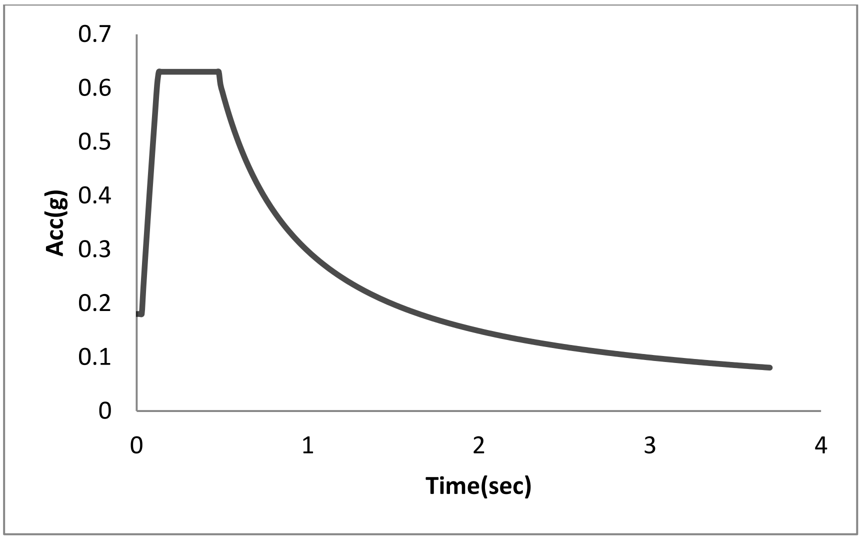


Figure 4.2 Target GRS for generating spectrum compatible time history

Using this response spectrum compatible time history is generated. For generating artificial time history from response spectrum for the first iteration, Fourier magnitude of ground motion is assumed to be equal to pseudo spectral velocity derived from target spectra. Then the signal is made non-stationary by multiplying output of inverse Fourier transform with an envelope function like Boore, Sarongi and Hart. And it is ensure that the signal is band passed in the frequency range of 0.1 Hz to 25 Hz because frequency below 0.1Hz gives very unrealistic displacement value of the earthquake and frequency above 25HZ have very less effect on the earthquake accelerograms.

The software used is Seismo-Artif, whose theory behind generation of the time history is given below

There are 4 types of method of generating time history which are

a. Synthetic Accelerogram Generation and Adjustment

b. Artifical Acclerogram Generation

c. Artifical Acclerogram Generation and Adjustment

d. Real Accelerogram Adjustment

a. Synthetic Accelerogram Generation and Adjustment

It is based on Hallodorson and Papageorgiou(2005) . Here at first artificial acceleration is defined from synthetic one (Gaussian white noise multiplied by sarogi and hart envelope) and then correction is applied in frequency domain .Such that

$$F(f)_{i+1} = F(f)_i \left[SRT(f) / SR(f)_i\right] \tag{33}$$

Where,

$F(f)_{i+1}$ =Value of acc. In frequency domain for current iteration

$F(f)_i$ = Value of acc. In frequency domain for previous iteration

$SRT(f)$ = Value of target spectrum for frequency f

$SR(f)_i$ = Value of response spectrum corresponding to acceleogram

b. Artificial Acclerogram Generation

In this method it defines each ground motion modifying the starting random process through use of envelope shape and power spectral density function.

Here artificial ground motion is input as

$$Z(t) = I(t)\sum A_n \sin(\omega_n t + \phi_n) \tag{34}$$

Here correction is made using PSDF

$$G(\omega)_{i+1} = G(\omega)_i \left(\frac{S_y}{S_y^i}\right)^2 \tag{35}$$

Where,

S_y= Target spectrum value and S_y^i= Computed response spectrum value

c. Artificial Acclerogram Generation and Adjustment

It is same as artificial accelerogram generation but correction is made in frequency domain. It have stable and good convergence rate with high number of iteration with result same as artificial accelerogram generation.

d. Real Accelerogram Adjustment

The real time accelerogram from real earthquake date are scaled down and up and corrected in frequency domain to give a match to the target spectrum.

For the project we used fourth method. one of the generated artificial earthquake .

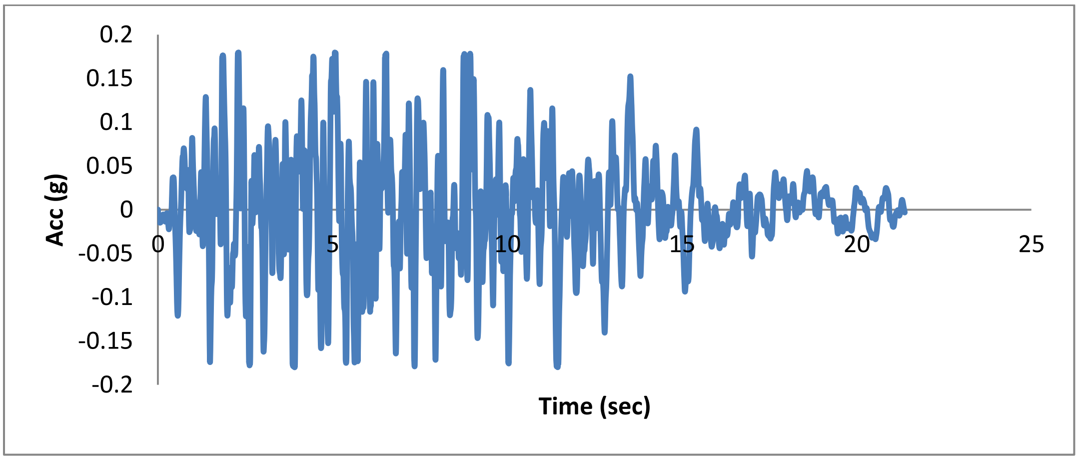


Figure 4.3 Gadarwara site spectrum compatible time history

As a rough check the PGA of the above time history is 0.18g which is the same as ZPA of the response spectrum of Gadarwara.

4.4 Linear modal time history analysis

The coupled equation of motion is

$$[M]\{\ddot{x}\}+[C]\{\dot{x}\}+[K]\{x\}=[F(t)] \tag{36}$$

Where,

$[M],[C],[K]$ = Mass, Damping and Stiffness matrices of building

$\{x\},\{\dot{x}\},\{\ddot{x}\}$ = Displacement, velocity and acceleration vectors

$[F(t)]$ = Forcing function vector for the model.

The uncoupling of equation can be derived using mode orthogonality relation

$\phi^T M\phi=\dot{M}$ and $\phi^T K\phi=\dot{K}$ where ϕ =mode shape vector

The coupled equation of motion is decoupled using principle of superposition of mode shape

$$x=\sum_{i=1}^{i=n}\phi_i \times q_i \tag{37}$$

Here ϕ_i is constant with time and q_i is a function of time .Hence when doing differentiation of x, ϕ is treated as constant and q is taken as variable.

The resultant uncoupled equation can be written in matrix form as

$$[\dot{M}]\{\ddot{q}\}+[\dot{C}]\{\dot{q}\}+[\dot{K}]\{q\}=[p_{eff}(t)] \tag{38}$$

$$\lfloor p_{eff}(t)\rfloor=[-\ddot{x}_g(t)\Gamma_r] \tag{39}$$

$$\Gamma_r=\{\phi\}^T[M]\{I\}/\dot{M} \tag{40}$$

$\Gamma_r =$ Mode participation factor

The uncoupled equation can be solved using various numerical techniques like

a. Interpolation of excitation function

b. Central difference method

c. Newmark Method

Our main aim lies in finding out acceleration from the uncoupled equation which is given by

$$\ddot{x} = \sum_{i=1}^{i=n} \phi_i \times \ddot{q}_i \tag{41}$$

Here, $\ddot{x}$ is a vector of size $(\mathrm{N} \times 1)$

$$\ddot{q}_i = (q_{i+1} - 2 \times q_i + q_{i-1}) / (\Delta t)^2 \tag{42}$$

For the construction of floor spectrum let z denotes the absolute displacement of floor

Then the equation of motion for floor is

$$\ddot{Z} + 2\xi\omega_n \dot{Z} + \omega_n{}^2 Z = \ddot{x}(t) \tag{43}$$

4.5 Linear direct integration time history analysis

Direct Integration Time History Analysis directly integrates the equation of motion without the use of modal superposition method. The various methods available in SAP2000 for Direct integration are:

(i) Newmark Method

(ii) Wilson Method

(iii) Collocation

(iv) Hiber-Hughes Taylor

(v) Chung and Hulbert

In this study Hiber-Hughes Taylor method has been used. For simple structure problems(whose solution converges easily), using Newmark method with $\beta = {}^{1}/_{4}$, gives constant average acceleration method, and for nonlinear time-history with convergence problem, HHT method with $0 < \alpha \leq -{}^{1}/_{3}$ gives good convergence. While applying HHT, using value of $\alpha = 0$, is same as average acceleration method, so using HHT will be sufficient for all problem.

The Newmark method is stable when $2\ \beta \geq \gamma \geq 1/2$, so very often we use $\beta = 1/4$. When $\beta = 1/6$, it gives linear acceleration method, which becomes unstable when $\Delta t /T > 2\sqrt{3}/(2\pi)$, where Δt is the time step and T is the shortest structural period for the given structural loading. The shortest period can be obtained from modal analysis.

Direct integration are sensitive to the length of the time step. As per AERB criteria the time step (Δt) in the system shall be small enough to give stability and convergence of the problem. As a rule it is stated that Δt should be small enough so that use of ½ Δt does not effect response of the solution by more than 10%than Δt. For commonly used methods, Δt values are listed in Table 7.

Table 7 Values for time step used in numerical integration

Method	Fraction of Shortest Period of interest
Houbolt	1/15
Newmark	1/10
Wilson	1/10

SAP2000 also recommends the use of HHT method. It uses a single parameter called alpha whose value lies between 0 and –(1/3). For alpha=0, gamma=0.5 and beta=0.25 it results in average acceleration method also known as trapezoidal rule. Using alpha=0 gives the high accuracy of all the methods above but it results in more vibration in big frequency modes. To damp the higher frequency mode using negative alpha is best.

In this method one can assign damping corresponding to frequency of the modes in two different modes and then the system creates damping matrix corresponding to Rayleigh damping i.e.

If M and K denotes the mass and stiffness matrix, and if ω_1, ξ_1 and ω_2, ξ_2 represents frequency and damping corresponding to two different modes under consideration then the damping matrix C is given by

$$C = \alpha M + \beta K \tag{44}$$

Where,

$$\alpha = 2\xi_1\omega_1 - \beta\omega_1^2 \tag{45}$$

$$\beta = \frac{2\xi_1\omega_1 - 2\xi_2\omega_2}{\omega_1^{\,2} - \omega_2^{\,2}} \tag{46}$$

The variation of damping matrix with frequency will be given by

$$\xi_n = \frac{\alpha}{2\omega_n} + \frac{\beta\omega_n}{2} \tag{47}$$

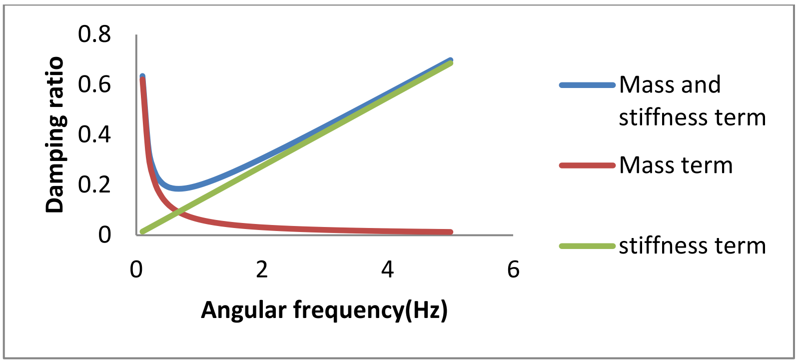


Figure 4.4 Variation of damping with frequency

In our model we have assigned two percent damping in first and third mode in x direction, hence as per Rayleigh damping matrix the higher mode are damped more.

5 RESULTS AND DISCUSSION

5.1 Dynamic characteristics

Here Table 8 shows time period and modal mass participation ratio of modes in building

Table 8 Time period and modal mass participation ratio of modes in building

Mode	Period(sec)	UX(%)	UY(%)	UZ(%)	SumUX	SumUY	SumUZ
1	1.545741	2.378	52.629	1.75E-05	2.378	52.629	1.75E-05
2	1.404274	47.431	6.538	3.17E-09	49.809	59.167	1.76E-05
3	1.172477	19.563	2.503	7.12E-06	69.372	61.67	2.47E-05
4	0.621064	21.726	1.001	6.31E-06	91.098	62.671	3.1E-05
5	0.59795	0.863	22.129	0.000452	91.961	84.8	0.000483
6	0.542396	0.023	1.355	0.000444	91.984	86.155	0.000926
7	0.523586	0.002768	0.000481	0.18	91.987	86.155	0.181
8	0.454858	0.134	0.118	0.001951	92.121	86.273	0.183
9	0.447165	5.31E-05	1.512	0.011	92.121	87.785	0.193
10	0.401694	0.211	0.97	0.033	92.333	88.755	0.226
11	0.378985	0.712	1.35	0.012	93.045	90.105	0.238
12	0.368522	0.291	0.575	2.59E-05	93.336	90.681	0.238

Where,

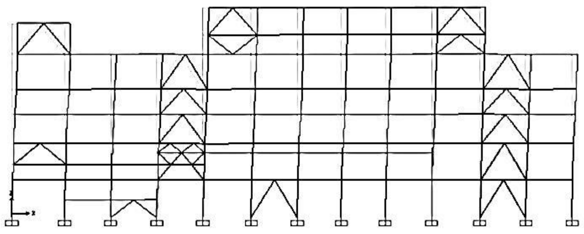
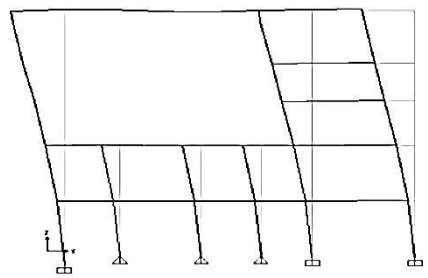
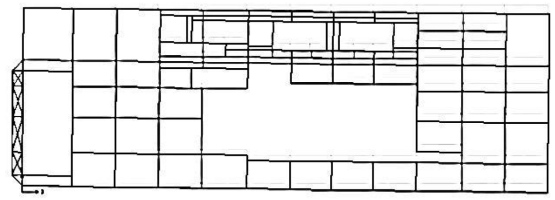
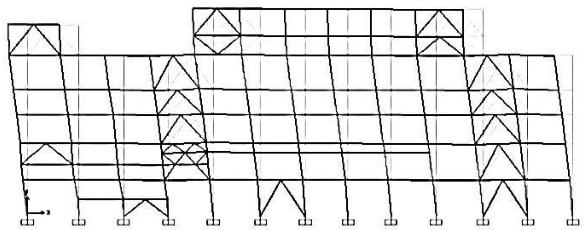
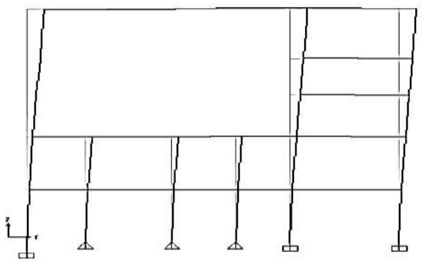
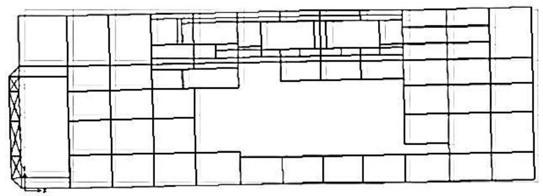
UX= Modal Mass Participation Percentage in x-direction.

UY= Modal Mass Participation Percentage in y-direction.

UZ= Modal Mass Participation Percentage in z-direction .

Here we also see that the modal mass participation percentage is greater than 90%, Hence the model can be used for the dynamic analysis. Mode shape of the structure can be seen from Figure 5.1 to 5.18 in Table 9.

Table 9 Modes Shape of the thermal power plant

 ***Figure 5.1 Mode 1 longitudinal view***	 ***Figure 5.2 Mode 1 transverse view***	 ***Figure 5.3 Mode 1 plan view***
 ***Figure 5.4 Mode 2 longitudinal view***	 ***Figure 5.5 Mode 2 transverse view***	 ***Figure 5.6 Mode 2 plan view***

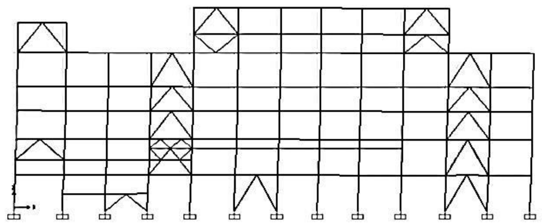


Figure 5.7 Mode 3 longitudinal view

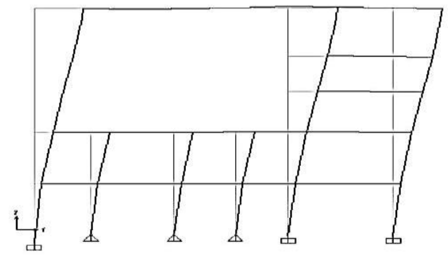


Figure 5.8 Mode 3 transverse view

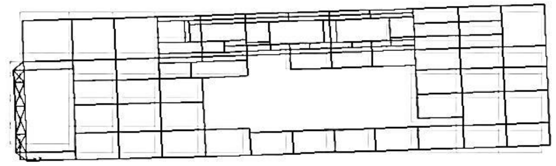


Figure 5.9 Mode 3 plan view

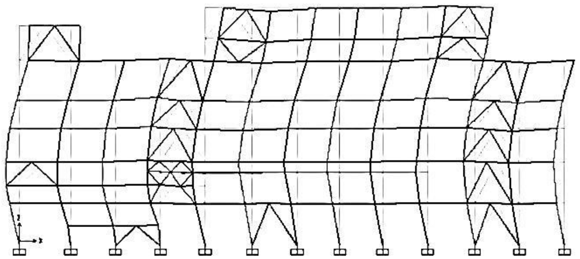


Figure 5.10 Mode 4 longitudinal view

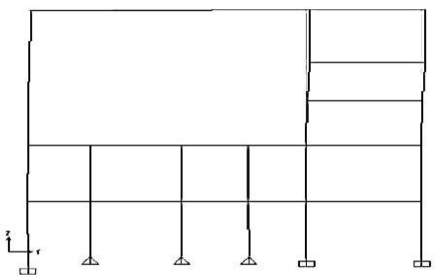


Figure 5.11 Mode 4 transverse view

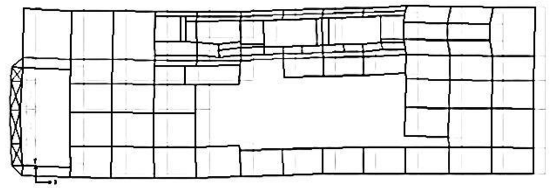


Figure 5.12 Mode 4 plan view

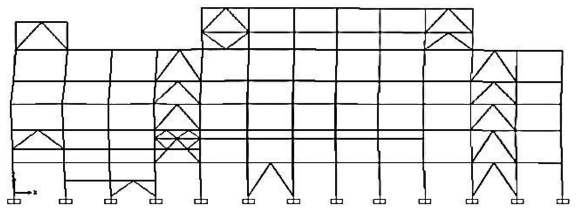


Figure 5.13 Mode 5 longitudinal view

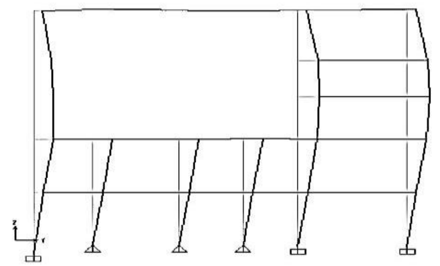


Figure 5.14 Mode 5 transverse view

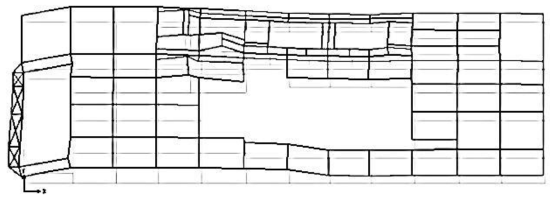


Figure 5.15 Mode 5 plan view

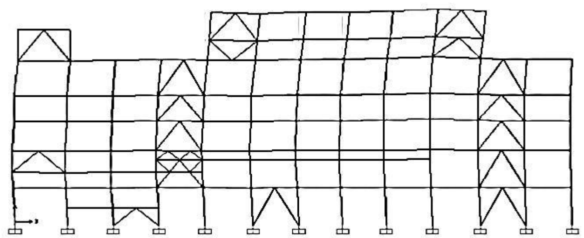


Figure 5.16 Mode 6 longitudinal view

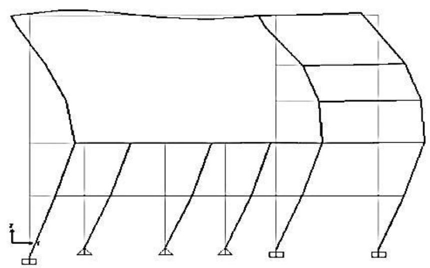


Figure 5.17 Mode 6 transverse view

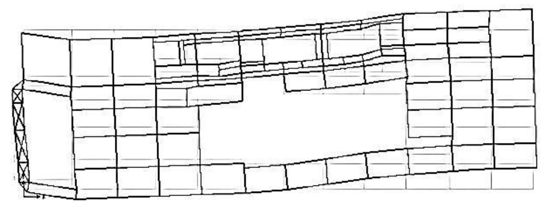


Figure 5.18 Mode 6 plan view

P- delta effect

Table 10 P-Delta effect on period of vibration in different modes

Mode No.	Time period (sec)	
	without P delta	with P delta
1	1.54	1.55
2	1.40	1.41
3	1.17	1.17
4	0.62	0.62
5	0.59	0.60
6	0.54	0.54
7	0.52	0.52
8	0.45	0.45
9	0.44	0.44
10	0.40	0.40
11	0.37	0.38
12	0.36	0.36

Here in Table 10 we see that due to P-delta effect there is not much effect on the period of the structure, hence effect of P-delta is negligible in the structure.

5.2 Linear dynamics response

Variation of PFA along height using Modal time history analysis (MTHA) from Table 11-14 is given in term of (g) for seven Time history.

Table 11 Variation of PFA along height for A-28 column

Height (m)	Peak Floor Acceleration (g)							Average PFA/PGA
	TH-1	TH -2	TH -3	TH -4	TH -5	TH -6	TH -7	
base	0.18	0.18	0.18	0.18	0.18	0.18	0.18	1.00
8.50	0.27	0.37	0.30	0.35	0.32	0.34	0.23	1.73
18.00	0.40	0.50	0.43	0.50	0.41	0.48	0.35	2.44
25.50	0.39	0.34	0.33	0.38	0.33	0.38	0.29	1.94
32.00	0.36	0.20	0.22	0.28	0.28	0.27	0.25	1.48
39.00	0.53	0.36	0.39	0.42	0.51	0.45	0.40	2.42
41.00	0.59	0.42	0.45	0.47	0.57	0.51	0.45	2.75

Table 12 Variation of PFA along height for A-16A column

Height (m)	Peak Floor Acceleration (g)							Average PFA/PGA
	TH-1	TH -2	TH -3	TH -4	TH -5	TH -6	TH -7	
base	0.18	0.18	0.18	0.18	0.18	0.18	0.18	1.00
8.50	0.26	0.32	0.27	0.30	0.30	0.30	0.22	1.56
18.00	0.37	0.44	0.39	0.42	0.38	0.43	0.33	2.20
25.50	0.36	0.26	0.29	0.34	0.30	0.31	0.24	1.67
32.00	0.35	0.19	0.23	0.27	0.27	0.27	0.26	1.45
39.00	0.51	0.34	0.39	0.41	0.50	0.43	0.39	2.35
41.00	0.55	0.41	0.42	0.41	0.52	0.48	0.42	2.5

Table 13 Variation of PFA along height for C-28 column

Height (m)	Peak Floor Acceleration (g)							Average PFA/PGA
	TH-1	TH -2	TH -3	TH -4	TH -5	TH -6	TH -7	
base	0.18	0.18	0.18	0.18	0.18	0.18	0.18	1.00
8.50	0.27	0.33	0.29	0.33	0.30	0.29	0.24	1.62
18.00	0.42	0.46	0.39	0.51	0.44	0.43	0.35	2.38
25.50	0.47	0.37	0.35	0.44	0.38	0.35	0.33	2.13
32.00	0.43	0.33	0.27	0.32	0.29	0.27	0.29	1.75
41.00	0.49	0.43	0.41	0.50	0.49	0.37	0.47	2.51

Table 14 Variation of PFA along height for C-16A column

Height (m)	Peak Floor Acceleration (g)							Average PFA/PGA
	TH-1	TH -2	TH -3	TH -4	TH -5	TH -6	TH -7	
base	0.18	0.18	0.18	0.18	0.18	0.18	0.18	1.00
8.50	0.23	0.29	0.26	0.26	0.28	0.26	0.21	1.41
18.00	0.39	0.39	0.34	0.42	0.38	0.35	0.39	2.11
25.50	0.43	0.30	0.31	0.36	0.32	0.28	0.30	1.82
32.00	0.42	0.32	0.28	0.32	0.29	0.26	0.30	1.74
41.00	0.49	0.42	0.42	0.47	0.49	0.37	0.47	2.49

Variation of PFA along height using direct integration time history analysis (DITHA) is given from Table 15-23 in term of (g) for seven Time history.

Table 15 Variation of PFA along height for A-28 column

Height (m)	Peak Floor Acceleration (g)							Average PFA/PGA
	TH-1	TH -2	TH -3	TH -4	TH -5	TH -6	TH -7	
base	0.18	0.18	0.18	0.18	0.18	0.18	0.18	1.00
8.50	0.38	0.41	0.41	0.36	0.35	0.35	0.29	2.01
18.00	0.40	0.49	0.49	0.52	0.44	0.49	0.38	2.55
25.50	0.41	0.33	0.33	0.42	0.36	0.39	0.32	2.03
32.00	0.36	0.19	0.19	0.30	0.32	0.27	0.27	1.51
39.00	0.52	0.38	0.38	0.42	0.50	0.45	0.39	2.41
41.00	0.57	0.44	0.44	0.47	0.56	0.52	0.45	2.73

Table 16 Variation of PFA along height for A-22 column

Height (m)	Peak Floor Acceleration (g)							Average PFA/PGA
	TH-1	TH -2	TH -3	TH -4	TH -5	TH -6	TH -7	
base	0.18	0.18	0.18	0.18	0.18	0.18	0.18	1.00
8.50	0.33	0.37	0.36	0.36	0.34	0.30	0.26	1.84
18.00	0.41	0.44	0.39	0.46	0.39	0.47	0.33	2.29
25.50	0.40	0.29	0.29	0.39	0.32	0.35	0.28	1.85
32.00	0.36	0.18	0.23	0.29	0.29	0.27	0.26	1.49
39.00	0.51	0.37	0.40	0.41	0.49	0.45	0.39	2.40
41.00	0.56	0.43	0.46	0.46	0.55	0.52	0.45	2.73

Table 17 Variation of PFA along height for A-16A column

Height (m)	Peak Floor Acceleration (g)							Average PFA/PGA
	TH-1	TH -2	TH -3	TH -4	TH -5	TH -6	TH -7	
base	0.18	0.18	0.18	0.18	0.18	0.18	0.18	1.00
8.50	0.34	0.35	0.34	0.36	0.34	0.29	0.27	1.82
18.00	0.42	0.42	0.38	0.46	0.40	0.47	0.35	2.30
25.50	0.40	0.25	0.28	0.37	0.30	0.31	0.27	1.74
32.00	0.36	0.18	0.24	0.29	0.29	0.27	0.26	1.49
39.00	0.50	0.35	0.39	0.41	0.49	0.44	0.39	2.36
41.00	0.56	0.41	0.45	0.45	0.55	0.50	0.44	2.66

__Table 18 Variation of PFA along height for B-28 column__

Height (m)	Peak Floor Acceleration (g)							Average PFA/PGA
	TH-1	TH -2	TH -3	TH -4	TH -5	TH -6	TH -7	
base	0.18	0.18	0.18	0.18	0.18	0.18	0.18	1.00
8.50	0.33	0.38	0.33	0.35	0.34	0.33	0.23	1.82
18.00	0.41	0.46	0.42	0.47	0.41	0.44	0.33	2.34
25.50	0.49	0.43	0.42	0.40	0.39	0.39	0.39	2.31
32.00	0.47	0.38	0.35	0.32	0.36	0.34	0.38	2.06
41.00	0.49	0.44	0.41	0.47	0.45	0.40	0.51	2.52

__Table 19 Variation of PFA along height for B-22 column__

Height (m)	Peak Floor Acceleration (g)							Average PFA/PGA
	TH-1	TH -2	TH -3	TH -4	TH -5	TH -6	TH -7	
base	0.18	0.18	0.18	0.18	0.18	0.18	0.18	1.00
8.50	0.30	0.35	0.39	0.36	0.36	0.29	0.26	1.83
18.00	0.43	0.45	0.40	0.48	0.39	0.41	0.35	2.31
25.50	0.48	0.35	0.38	0.36	0.34	0.34	0.31	2.03
32.00	0.44	0.34	0.30	0.28	0.33	0.32	0.32	1.84
41.00	0.50	0.43	0.40	0.46	0.42	0.40	0.49	2.46

Table 20 Variation of PFA along height for B-16A column

Height (m)	Peak Floor Acceleration (g)							Average PFA/PGA
	TH-1	TH -2	TH -3	TH -4	TH -5	TH -6	TH -7	
base	0.18	0.18	0.18	0.18	0.18	0.18	0.18	1.00
8.50	0.31	0.35	0.36	0.35	0.34	0.26	0.26	1.76
18.00	0.39	0.42	0.35	0.43	0.37	0.36	0.30	2.08
25.50	0.43	0.31	0.33	0.34	0.32	0.30	0.30	1.85
32.00	0.43	0.23	0.32	0.29	0.30	0.33	0.33	1.77
41.00	0.49	0.43	0.40	0.45	0.43	0.39	0.48	2.44

Table 21 Variation of PFA along height for C-28 column

Height (m)	Peak Floor Acceleration (g)							Average PFA/PGA
	TH-1	TH -2	TH -3	TH -4	TH -5	TH -6	TH -7	
base	0.18	0.18	0.18	0.18	0.18	0.18	0.18	1.00
8.50	0.30	0.33	0.30	0.37	0.40	0.32	0.24	1.80
18.00	0.42	0.45	0.40	0.50	0.40	0.43	0.34	2.33
25.50	0.47	0.40	0.41	0.42	0.35	0.37	0.38	2.23
32.00	0.45	0.38	0.32	0.33	0.31	0.36	0.36	2.00
41.00	0.49	0.44	0.43	0.52	0.50	0.42	0.55	2.67

Table 22 Variation of PFA along height for C-22 column

Height (m)	Peak Floor Acceleration (g)							Average PFA/PGA
	TH-1	TH -2	TH -3	TH -4	TH -5	TH -6	TH -7	
base	0.18	0.18	0.18	0.18	0.18	0.18	0.18	1.00
8.50	0.26	0.29	0.29	0.29	0.27	0.24	0.26	1.51
18.00	0.41	0.45	0.39	0.48	0.41	0.41	0.33	2.29
25.50	0.44	0.33	0.39	0.39	0.34	0.32	0.30	1.99
32.00	0.44	0.38	0.34	0.33	0.31	0.36	0.35	2.00
41.00	0.48	0.44	0.43	0.50	0.49	0.40	0.51	2.58

Table 23 Variation of PFA along height for C-16A column

Height (m)	Peak Floor Acceleration (g)							Average PFA/PGA
	TH-1	TH -2	TH -3	TH -4	TH -5	TH -6	TH -7	
base	0.18	0.18	0.18	0.18	0.18	0.18	0.18	1.00
8.50	0.28	0.33	0.36	0.34	0.32	0.25	0.27	1.72
18.00	0.40	0.42	0.34	0.45	0.40	0.37	0.33	2.14
25.50	0.42	0.31	0.34	0.36	0.33	0.29	0.33	1.89
32.00	0.42	0.34	0.30	0.33	0.31	0.31	0.31	1.83
41.00	0.49	0.43	0.43	0.49	0.48	0.39	0.51	2.56

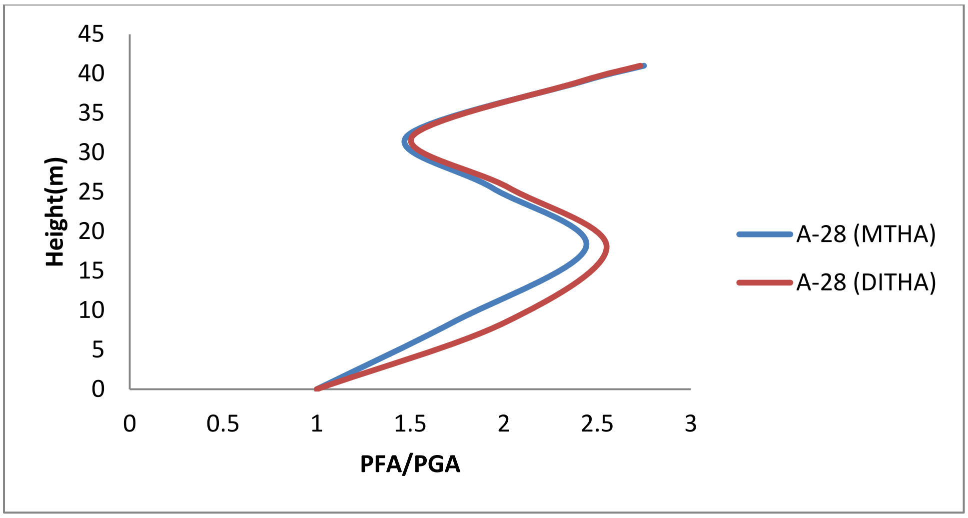


Figure 5.19 Comparision of MTHA and DITHA for column A-28

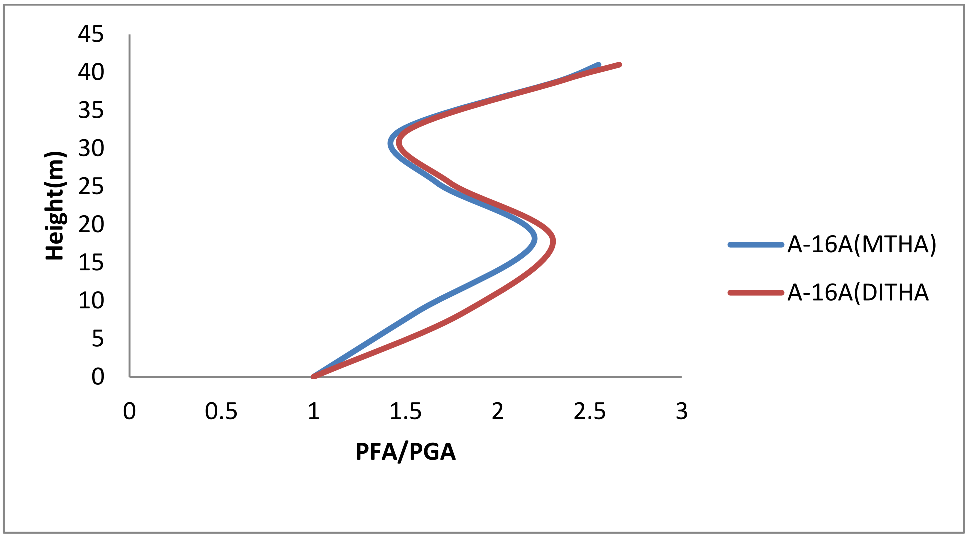


Figure 5.20 Comparision of MTHA and DITHA for column A-16A

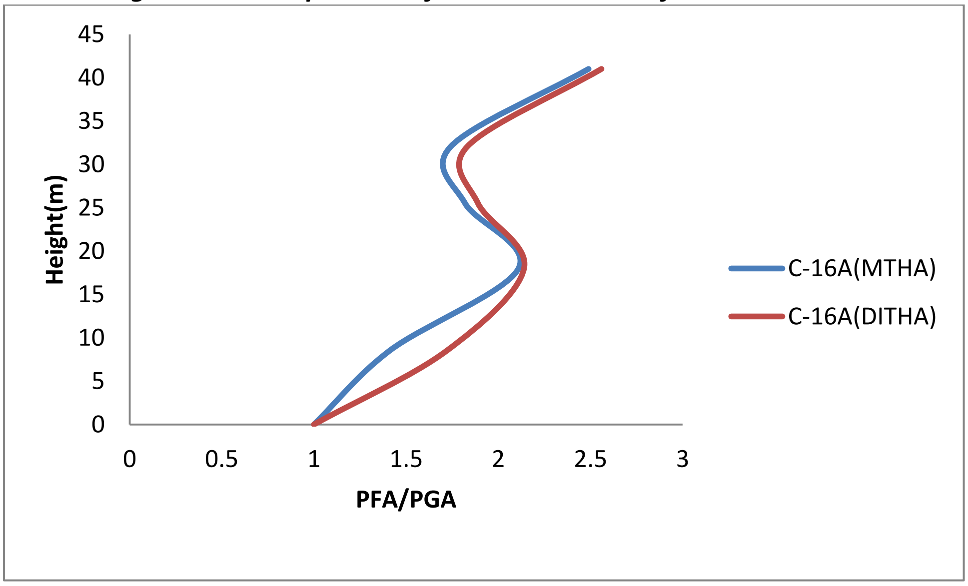


Figure 5.21 Comparision of MTHA and DITHA for column C-16A

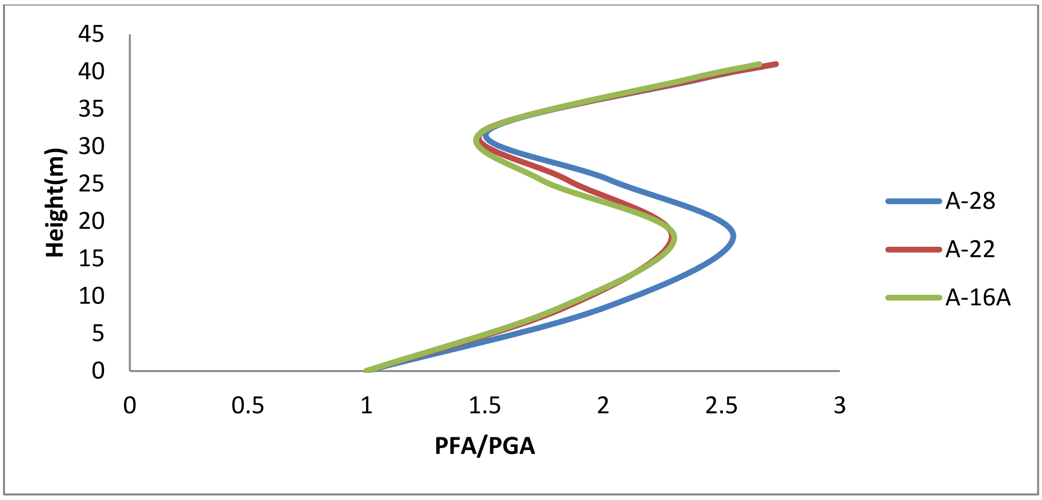


Figure 5.22 Comparision of response in A- row columns

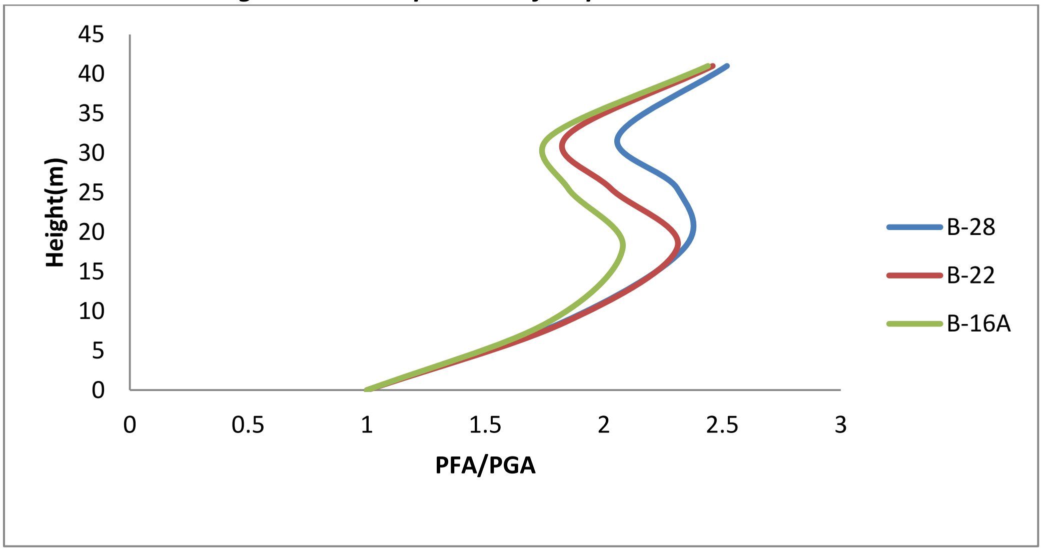


Figure 5.23 Comparision of response in B-row columns

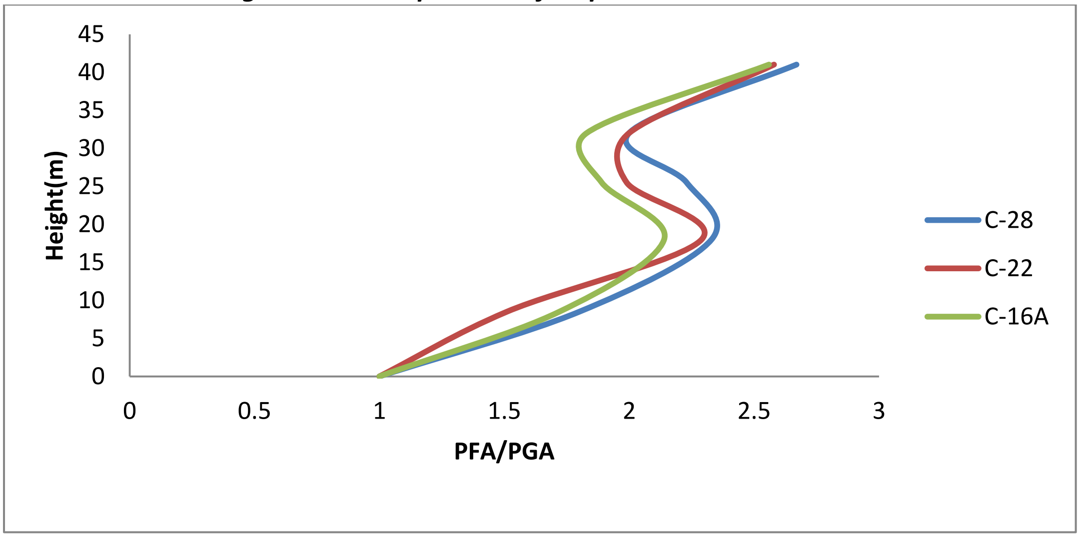


Figure 5.24 Comparision of response in C-row columns

Here from Figure 5.19 to Figure 5.21 we can conclude that direct integration time history analysis with Rayleigh damping gives slightly more response that Modal time history analysis with constant damping for the given structure.

Incase of Rayleigh damping 2% damping is given in first and third mode in x direction whereas in linear modal time history analysis constant damping of 2% is assigned.

For Table 15 to Table 22 and from Figure 5.22 to Figure 5.24 it is clear that A row have maximum PFA/PGA value at roof followed by C and B row which can be seen from Table 24 also.

Table 24 Value of PFA/PGA at roof (41m) of thermal power plant

A-Row		B- Row		C- Row	
Column Name	PFA/PGA at Roof (41m)	Column Name	PFA/PGA at Roof (41m)	Column Name	PFA/PGA at Roof (41m)
A-28	2.73	B-28	2.52	C-28	2.67
A-22	2.73	B-22	2.46	C-22	2.58
A-16A	2.66	B-16A	2.44	C-16A	2.56

Studying Value from Table 15 to Table 22 and observing the Figure 5.22 to Figure 5.24 it is clear that the variation of PFA/PGA is not linear as defined by the code. Here we see that PFA is increasing from base level to 18m and then decreasing from 18m to 32m and then again increasing upto roof level.The value of PFA/PGA at 18m height is given in Table 25.

Table 25 Value of PFA/PGA at 18m of thermal power plant

A-Row		B- Row		C- Row	
Column Name	PFA/PGA at Roof (41m)	Column Name	PFA/PGA at Roof (41m)	Column Name	PFA/PGA at Roof (41m)
A-28	2.55	B-28	2.34	C-28	2.33
A-22	2.29	B-22	2.31	C-22	2.29
A-16A	2.30	B-16A	2.08	C-16A	2.14

5.3 Response of deaerator

Both time history response and response spectrum response is shown below

As rigid diaphragm is not assigned to the model hence response of the different nodes lying in the same plane is not same. Hence response spectrum for different nodes is average to finally give the Floor response spectrum of the De-aerator.

De-aerator is located at the height of 41m. It is a very heavy Equipment and hence need to be design for earthquake. It is holded together by five beam which are in transverse direction containing serial no. from B 19-23 and C 19-23 (Figure 5.25). Hence there are 10 nodes. Seven set of time history in run in the model and at this 10 points response spectrum is generated for each time history and then individual is average and finally average of spectrum for all time history is done.

Time history Response

Time history response is given from Figure 5.26 to Figure 5.29

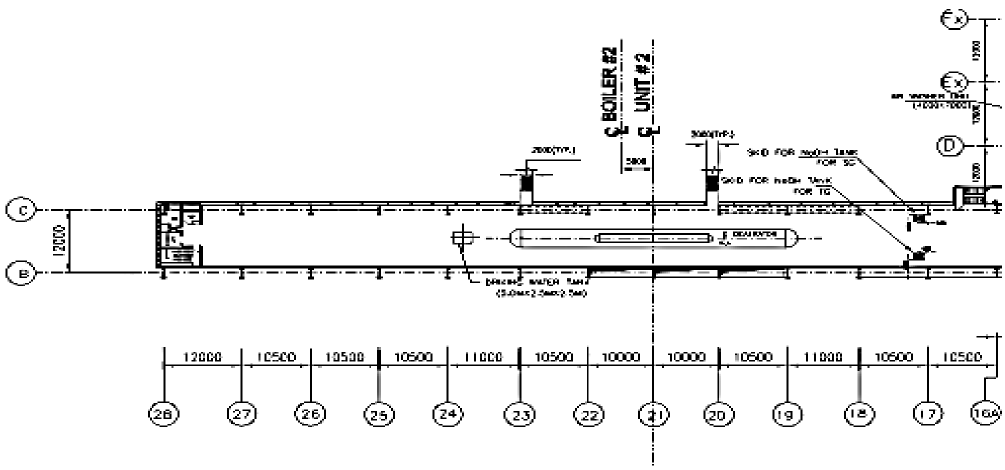


Figure 5.25 Plan at 41m

Here in the above figure 5.25 the deaerator machine is located in the nodes from B19-B23 and C19-C23.

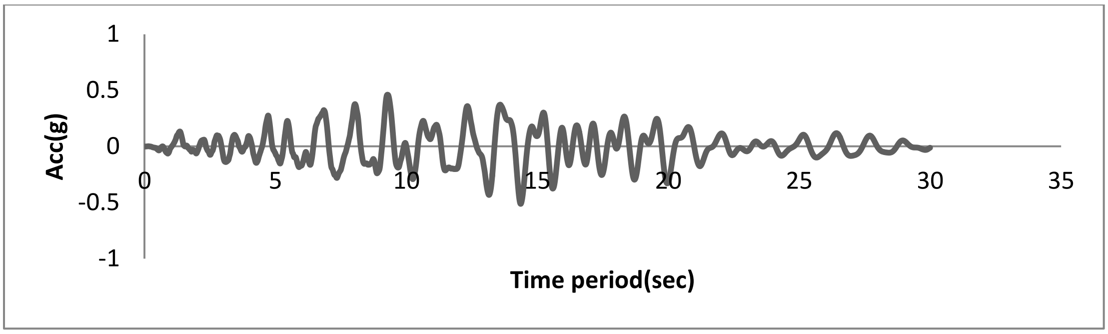


Figure 5.26 Absolute acceleration time history response of node B23 for TH-1

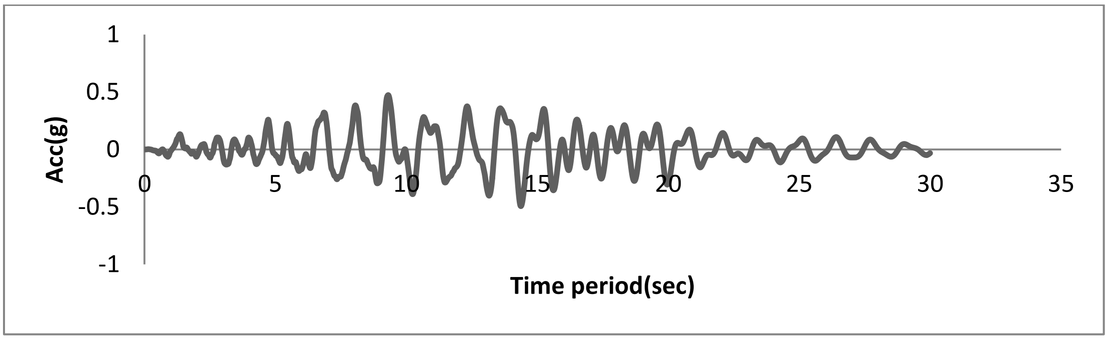


Figure 5.27 Absolute acceleration time history response of node C-21 for TH-1

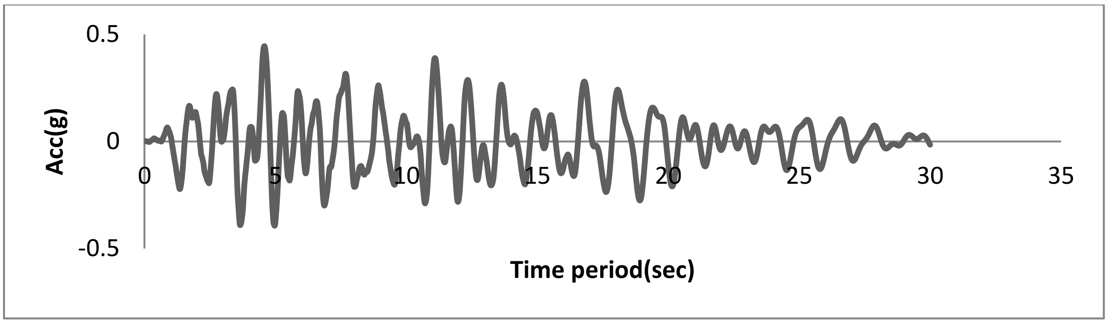


Figure 5.28 Absolute acceleration time history response of nodeB-23 for TH-2

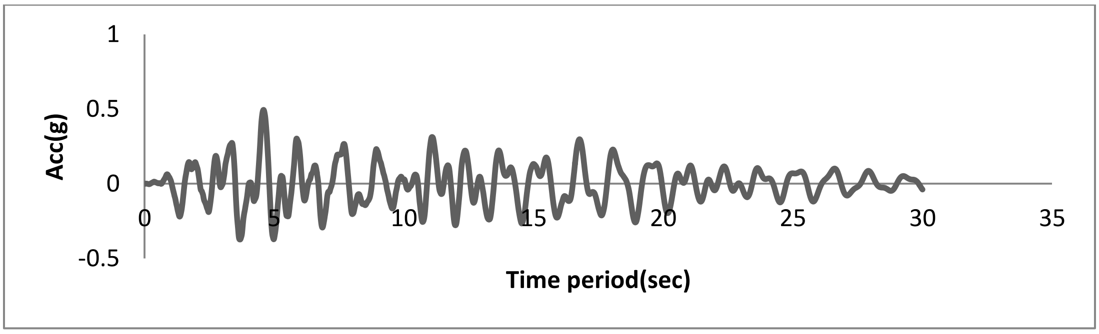


Figure 5.29 Absolute acceleration time history response of node C-21 for TH-2

Response Spectrum

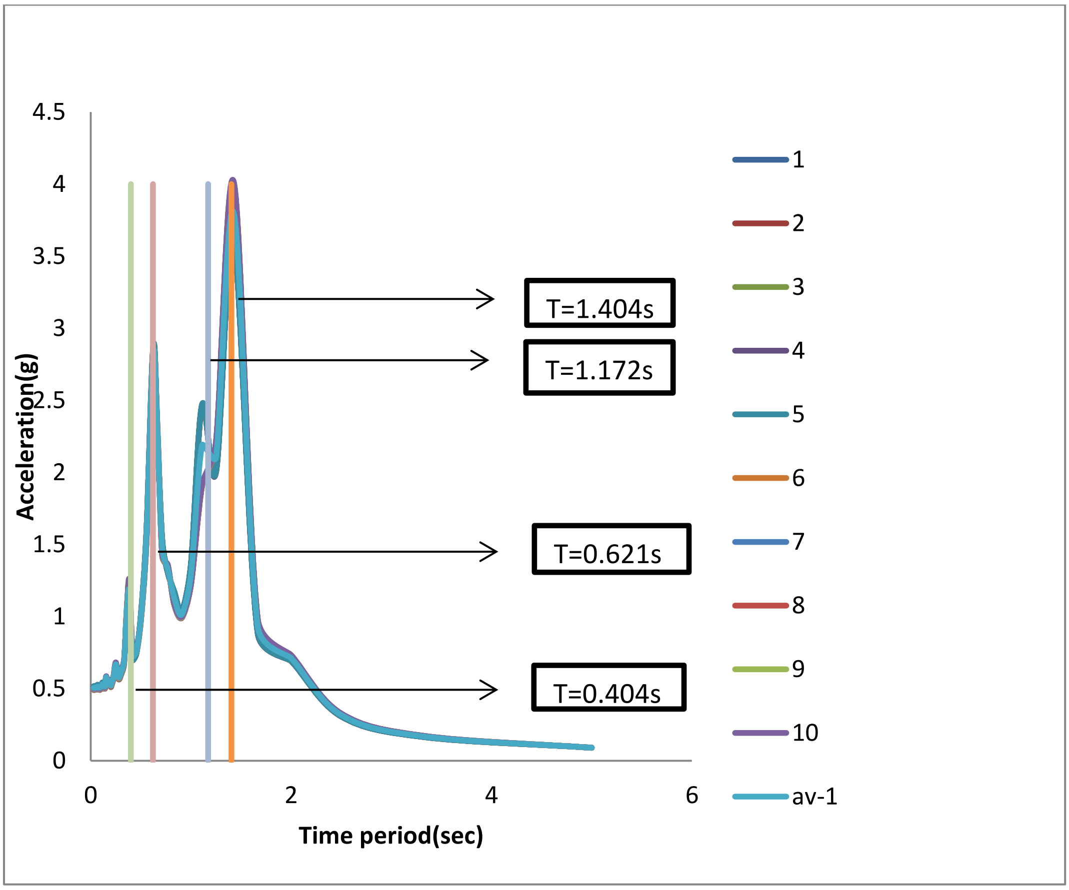


Figure 5.30 Response spectra for deaerator at different connecting nodes for TH-1

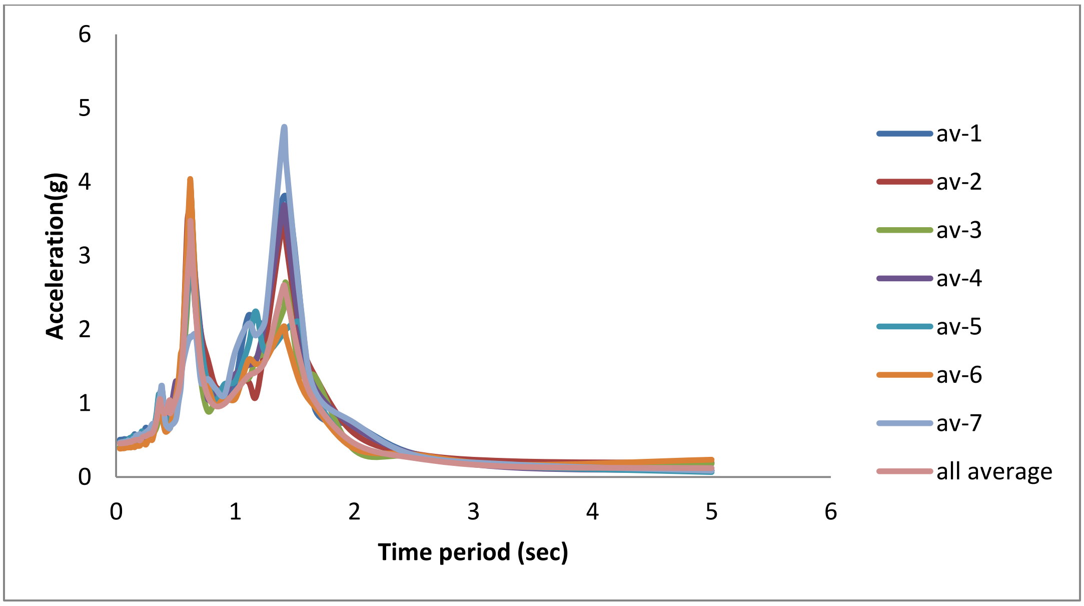


Figure 5.31 Average deaerator response for the seven time histories

Table 26 Peak acceleration value for different periods of the structure

S. No	Acceleration Value (g)			
	T=1.404 sec	T=1.172 sec	T=0.621 sec	T=0.404 sec
TH 1	3.796	2.189	2.830	0.728
TH 2	3.155	1.287	3.934	0.761
TH 3	2.602	1.334	3.913	0.777
TH 4	3.423	1.516	3.537	0.946
TH 5	2.003	1.789	3.014	0.775
TH 6	1.879	1.586	3.981	0.609
TH 7	4.23	2.076	2.512	0.711

Here from Table 26 and Figure 5.32 we can conclude that different time history even of same response spectrum gives completely different magnification for the different time period of the structure but the highest amplification usually occurs in the considered time period for the construction of Rayleigh damping matrix.

As in model 2% damping is assigned in T=1.404 sec and T=0.621 sec we here see that the highest amplification usually occurs in either of those two value.

Comparisons of coupled and decoupled floor response spectrum.

The mass of the deaerator is 157.5 ton and the mass of the structure excluding secondary system is 15824.2 ton, which makes the mass ratio as 0.00995 which is less than 0.01. Here in Figure 5.32 to Figure5.38 we are comparing the floor response of coupled and decoupled analysis of the deaerator for the set of seven time history. As in the Figure 5.30 we can see that the response of the different nodes at 41m are same, hence we take only one node (i.e. B-19) response for the comparision.

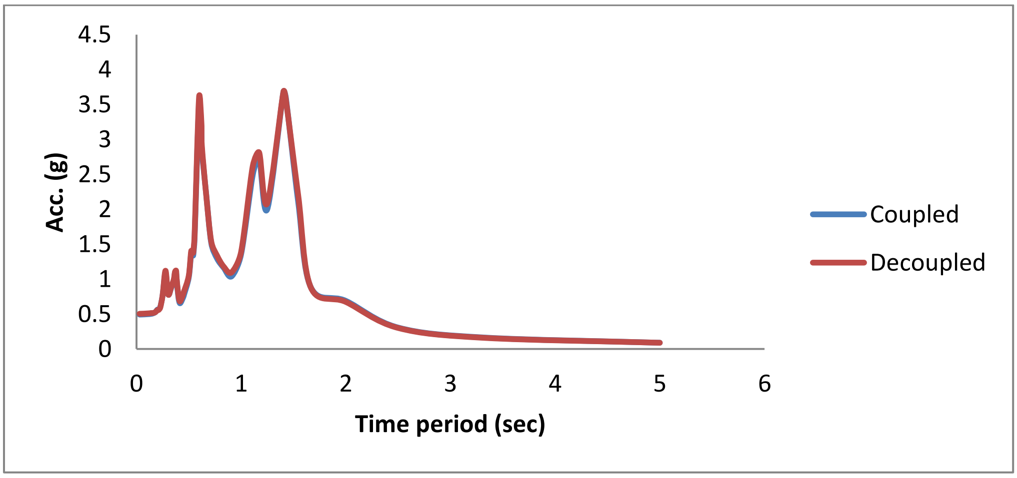


Figure 5.32 FRS for B-19 for TH-1

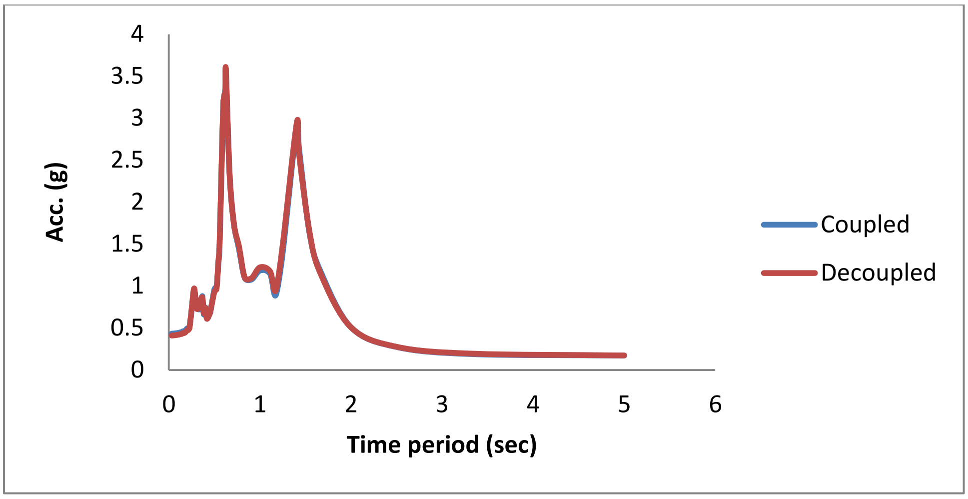


Figure 5.33 FRS for B-19 for TH-2

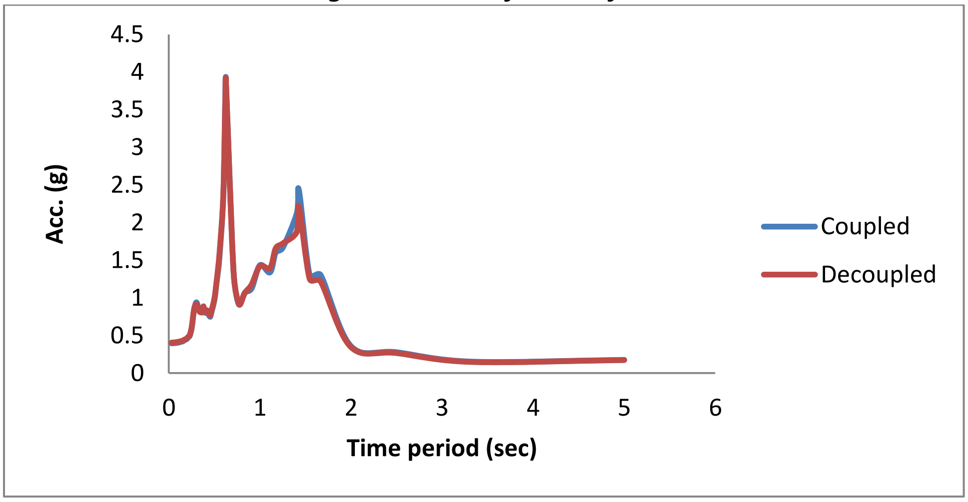


Figure 5.34 FRS for B-19 for TH-3

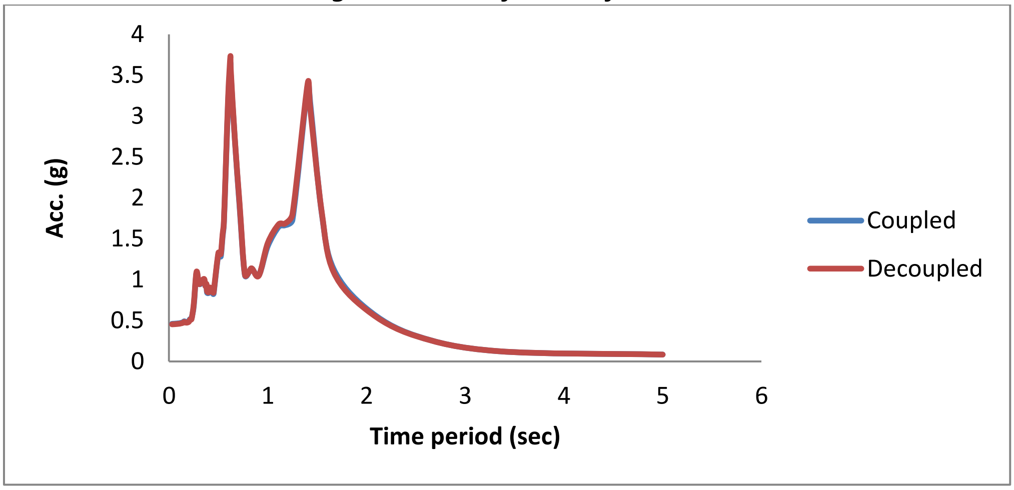


Figure 5.35 FRS for B-19 for TH-4

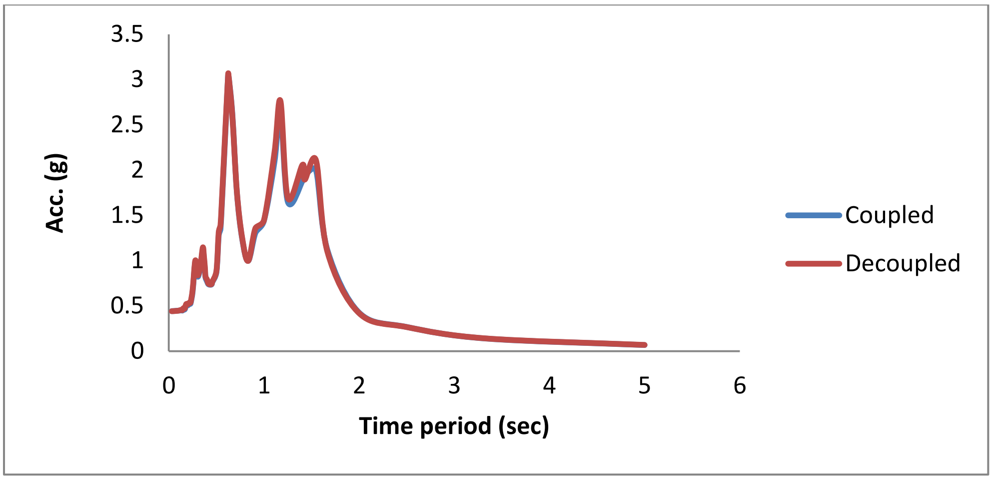


Figure 5.36 FRS for B-19 for TH-5

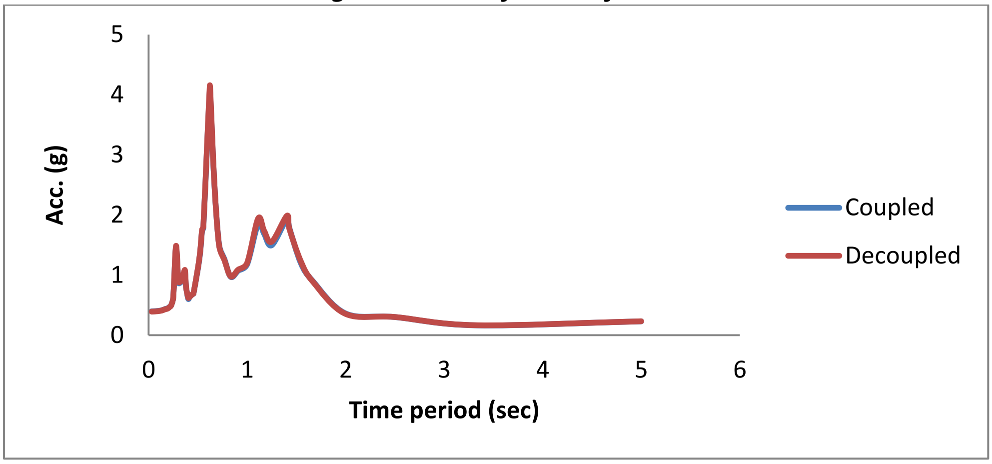


Figure 5.37 FRS for B-19 for TH-6

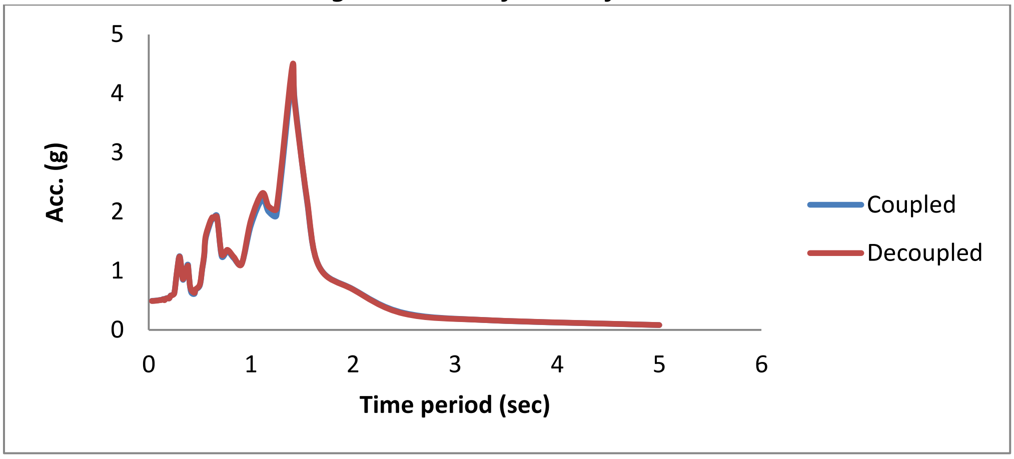


Figure 5.38 FRS for B-19 for TH-7

Here from Figure 5.32 to Figure 5.36 we see that coupled and decoupled analysis gives the same response. Maximum peak occurs in either first mode or third mode.This can be explained with the help of Fourier amplitude of the frequency of the given time history used for the analysis.

Table 27 Fourier amplitude of input ground motion for first and third mode in x-direction

Time history	Time period	Fourier amplitude of input ground acceleration for the given time period	Peak acceleration(g)
TH-1	1.404	0.0801	3.59
	0.621	0.0841	3.68
TH-2	1.404	0.0830	3.58
	0.621	0.0588	2.94
TH-3	1.404	0.0997	3.91
	0.621	0.0588	2.20
TH-4	1.404	0.1169	3.99
	0.621	0.0729	3.40
TH-5	1.404	0.0700	3.02
	0.621	0.0263	1.21
TH-6	1.404	0.1170	4.15
	0.621	0.0417	1.99
TH-7	1.404	0.0674	2.51
	0.621	0.1021	3.98

From Table 27 it is clear that whichever period have more Fourier amplitude that particular period have higher amplification in the response spectrum.

5.4 Inter-storey drift

The permissible inter storey drift as per IS 1893 is 0.4%. Inter- storey drift are calculated in both x and y direction for Eq-x and Eq-y force from response spectrum.

Table 28 Inter storey drift along x direction along A, B and C row

A-16A			B-16A			C-16A		
Height(m)	Drift(m)	Drift(%)	Height(m)	Drift(m)	Drift(%)	Height(m)	Drift(m)	Drift(%)
11	0.0252	0.229	9.7	0.0252	0.260	9.7	0.0256	0.264
9.5	0.0449	0.207	9.5	0.0483	0.243	9.5	0.0518	0.276
7.5	0.0558	0.145	7.5	0.0652	0.225	7.5	0.0719	0.268
6.5	0.0708	0.231	6.5	0.0839	0.288	6.5	0.0932	0.328
8.8	0.0984	0.314	9	0.1099	0.289	9	0.1226	0.327

From the Table 28 nowhere the structure inter-storey drift is exceeding the prescribed limit (0.4%) for earthquake in x direction.

Table 29 Inter storey drift along y direction along A, B and C row

A-16A			B-16A			C-16A		
Height(m)	Drift(m)	Drift(%)	Height(m)	Drift(m)	Drift(%)	Height(m)	Drift(m)	Drift(%)
11	0.0268	0.244	9.7	0.0265	0.273	9.7	0.0263	0.271
9.5	0.0601	0.351	9.5	0.0595	0.347	9.5	0.0605	0.360
7.5	0.0895	0.392	7.5	0.0878	0.377	7.5	0.0871	0.355
6.5	0.1117	0.342	6.5	0.112	0.372	6.5	0.109	0.337
8.8	0.1411	0.334	9	0.141	0.322	9	0.1421	0.368

Table 29 confirms the structure inter-storey drift is not exceeding the prescribed limit of 0.4% for earthquake in y direction.

6 CONCLUSION

In this Dissertation an under construction thermal power plant located in Gadarwara is modelled using SAP2000. Different types of joints and members are modelled. Moment connection joint are model as rigid joint, shear joint is modelled as pinned joint having M2 and M3 as free DOFs. Modelling of Roof Girder is assigned as tapered section. Different types of loading like piping load, Equipment load, Dead load, Live load are assigned as per details provided by NTPC.

Site specific response spectrum, available for the project site, prepared by IIT Roorkee has been used for generation of spectrum compatible time histories. A set of seven time histories has been used as input to calculate the response of the deaerator. Floor response Spectrum and time history response have been obtained using a linear time history analysis. The main conclusions of the study are:

a) As diaphragm is modelled using shell elements, it has been observed that the different nodes in the same floor have different response, however, the difference is negligibly small.

b) The variation of peak floor acceleration along the height is not linear and is dependent on the modal parameters of the structure such as modal frequency and modal mass participation. For the considered thermal power plant building, the value of PFA increases up to 18 m, then decreases from 18 m to 32 m, and again increases to the maximum at the top.

c) The maximum value of PFA at the roof level (41m) lies between 2.5 to 2.7.

d) The highest amplification of acceleration in floor response spectra usually occurs in one of the two time period which are assigned in the Rayleigh damping for the formation of damping matrix. The question of which period gets higher magnification can be answered by seeing the Fourier amplitude of the input ground motion at the assigned two period for the formation of damping matrix.

e) The floor response spectrum magnification factor at the period of the structure are different for the different time history that is generated from the single response spectrum.

f) The coupled and decoupled approach shows same response of the deaerator as mass ratio is very low.

REFERENCES

[1] NTPC, "wikimapia.org," [Online]. Available: http://wikimapia.org/25157794/Gadarwara-Super-Thermal-Power-Plant-under-construction. [Accessed November 2015].

[2] Electrical4u, "http://www.electrical4u.com," [Online]. Available: http://www.electrical4u.com/thermal-power-generation-plant-or-thermal-power-station/. [Accessed november 2015].

[3] BGR Energy system limited, "http://www.bgrcorp.com/eed_dearators.php," [Online]. Available: http://www.bgrcorp.com/eed_dearators.php. [Accessed 23 April 2016].

[4] Wikipedia, "https://en.wikipedia.org," [Online]. Available: https://en.wikipedia.org/wiki/Deaerator. [Accessed November 2015].

[5] W. Jiang, B. Li, M. D. Pandey and W. C. Xie, "Generate floor response spectra: Part 1. Direct spectra-to-spectra method," *Nuclear Engineering and Design,* vol. IV, no. 293, pp. 525-546, 2015.

[6] C. Duff, "Simplified method for development of earthquake ground and floor response spectra for nuclear power plant design.," Atomic Energy of Canada Limited, Sheridan Park, Ontario, 1975.

[7] M. Singh, "Generation of seismic floor spectra," *Eng. Mech. Div,* vol. v, no. 101, pp. 593-607, 1975.

[8] Y. Yasui, J. Yoshihara, T. Takeda and A. Miyamoto, "Direct generation method for floor response spectra," in *Proceedings of the 12th International Conference on Structural Mechanics in Reactor Technology,* Chicago, 1993.

[9] Y. An, Y. Jiang and B. Ly, "Direct generation of RRS from FRS," in *Proceedings of the 22nd International Conference on Structural Mechanics in Reactor Technology.*, 2013.

[10] A. Der Kiureghian, J. Sackman and B. Nour-omid, "Dynamic analysis of light equipment in structures: response to stochastic input.," *Eng. Mech.,* vol. i, no. 109, pp. 90-110, 1983.

[11] V. Vukobratovi and P. Fajfar, “A method for direct determination of inelastic floor response spectra,” *Engg. Struct.*, vol. 15, no. 6, pp. 1536-1561, 2012.

[12] M. Pinkawa, B. Hoffmeister and M. Feldmann, “A critical review of current approaches on the determination of seismic force demands on nonstructural components,” in *Proceedings of the 9th International Conference on Structual Dynamics*, Porto, Portugal, 2014.

[13] G. Mondal and S. K. Jain, “Design of non-structual element for building: A review of codal provision,” *Indian Concrete Journal*, vol. II, no. 4, pp. 22-28, 2005.

[14] European committee for standardisation, Design of structures for earthquake resistant-part1: General rules,seismic actions and rules for buildings, Brussels: European committee for standardisation, 2003.

[15] Uniform Building code, UBC, California: International Conference on Building, 1997.

[16] International Building code, IBC, USA: International Code Council, 2003.

[17] N. 4203:1992, Code of practice for general structural design and design loading for buildings, Wellington, New Zealand: code of practice Standards Association of New Zealand, 1992.

[18] S. K. Jain and C. V. R. Murty, Proposed Draft Provision and Commentary on Indian Seismic Code IS 1893 (part 1), Kanpur: IITK – GSDMA, 2010.

[19] I.S (part1), Indian standard criteria for earthquake resistance design ofstructures part 1 General provision and buildings (fifth revision), NewDelhi: Bureau of Indian Standards, 2002.

[20] SIA 261, Einwirkungen auf Tragwerke, Schweizer Norm SN 505 261, Switzerland, 2003.

[21] ASCE 7-05, Minimum design loads for buildings and other structures, Newyork: American Society of Civil Engineers, 2005.

[22] N. C. Tsai and W. S. Tseng, Standard Seismic Design Spectra for Nuclear Plant Equipment, California: North-Holland Publishing Company, 1978, pp. 481-488.

[23] F. X. Flores, D. L. Garcia and F. A. Charney, “Assessment of floor accelerations in special steel moment frames,” *Journal of Constructional Steel Research,* vol. IV, no. 106, pp. 154-165, 2015.

[24] Atomic Energy Regulatory Board, Seismic Quantification of Structures, System and Components of Pressurised Heavy Water Reactor, Mumbai, India: Govt. of India, 2009.

[25] R. Villaverde, “Simple method to estimate the seismic nonlinear response of non-structural component in building,” *Engg. Struct.*, vol. 28, no. 8, pp. 1209-1221, 2006.

[26] C. Yongqi and T. T. Soong, “Seismic response of secondary system,” *Engg. Struct.,* vol. 10, no. 4, pp. 218-228, 1998.