

Chapter 3. Estimation of Earthquake Load Effects

3.1 Introduction

Seismic action on chimneys forms an additional source of natural loads on the chimney.

Seismic action or the earthquake is a short and strong upheaval of the ground. This naturally is the cause for loads on any structure. Any structure under seismic loading is subjected to cyclical loading for a short period of time.

An earthquake is described by its intensity and its epicenter.

The intensity of an earthquake at a place is a measure of the degree of shaking caused during the earthquake and thus characterizes the effect of the earthquake. Most of the study of earthquakes up to the beginning of the twentieth century dealt with the effects of earthquakes and to quantitatively describe these effects a number of intensity scales were introduced. Initially there was the Rossi-Forel scale that had ten divisions. In 1888 Mercalli proposed a scale with 12 subdivisions to permit a clear distinction in shocks of extreme intensity. After a number of changes the Modified Mercalli scale or simply the MM scale is generally used by engineers today. Another revision made in 1956 to the MM scale by Richter is also in use.

The focus is the source for the propagation of seismic waves. It is also called the hypocenter. The depth of the focus from the surface of the earth directly above is referred to as the focal depth. The point on the earth's surface directly above the focus is known as the epicenter.

The structure experiences cyclic loading during the process of seismic action. This causes energy to build up in the system leading to its collapse. The friction with air, friction between particles that constitute the structure, friction at junctions of structural elements, yielding of the structural material and other processes of energy dissipation depress the amplitude of motion of a vibrating structure and the vibrations die out in course of time. When such internal and or external friction fully dissipates the energy of the structural system during its motion from a displaced position to its initial position of rest, inhibiting oscillations of the structure, the structure is said to be critically damped.

Thus the damping beyond which motion will not be oscillatory is called ‘critical damping’.

The effect of energy dissipation in reducing successive amplitude of vibrations of a structure from the position of static equilibrium is called damping and is expressed as a percentage of critical damping.

There are other terms that are important with respect to seismic analysis. During earthquakes there occurs a state in saturated cohesion less soil where in the effective shear strength is reduced to a negligible value, for all engineering purposes. Under this condition the soil tends to behave like a fluid mass.

A system is said to be vibrating in its normal mode or principal mode when all its masses attain maximum values of displacement simultaneously and they also pass through the equilibrium positions simultaneously. When a system is vibrating in its normal mode, the amplitude of the masses at any particular time expressed as a ratio of the amplitude of one of the masses is known as the mode shape coefficient.

During an earthquake ground vibrates (moves) in all directions. The horizontal component of the ground motion is generally more intense than that of the vertical components during strong earthquakes. The ground motion is generally random in nature and generally the random peaks of various directions may not occur simultaneously. Hence for design purposes, at one time, it is assumed that only the horizontal component acts in any one direction. All structures are designed to withstand their own weight. This could be deemed as though a vertical acceleration of $1g$ is applied to the various masses of the system. Since the design vertical forces proposed in the codes are small as compared to the acceleration of 1 gravity, the same emphasis has not been given to the vertical forces as compared to the horizontal forces. However for structures where stability is a criterion it may become necessary to take into account these vertical forces.

3.2 Estimation of loads

The seismic action is described by means of a standardized acceleration response spectrum. The CICIND code suggests a general response spectra. The response spectra is a relation between the maximum effective peak ground acceleration at the location of the

chimney. This is in relation with the natural time period of the structure and the soil type existing at the site.

The movement of the chimney is found by calculating the first few mode shapes by modal analysis of the chimney. The result of such a modal analysis will yield the values for the deflection, the shear force and the moment.

The modal analysis can determine the functions of the deflection, shear and the moment only up to a constant factor. Thus if the mode shape calculated is known, then a constant times the mode shape too is a possible solution.

Hence the actual value of the shear force or the bending moment is found by multiplying the normalized response with a scaling factor.

Hence if \bar{u} is the value of the normalized mode shape then the true mode shape is given by

$$u_i = \bar{u}_i N_i \quad (3.1)$$

Where they refer to the i^{th} mode of vibration, and N_i is the scaling factor. The scaling factor is determined by the following equation.

$$N_i = \frac{p_i T_i^2}{4\pi^2} a_s(T_i) \quad (3.2)$$

The a_s is the response function described earlier. The value of p_i is obtained from

$$p_i = \frac{\int_0^h \bar{u}_i(z) m(z) dz}{\int_0^h \bar{u}_i^2(z) m(z) dz} \quad (3.3)$$

The code also assumes the vertical movements to result in a value of resultants that are 0.3 times the horizontal forces.

The ACI code also assumes the vertical component to be negligible with respect to the horizontal one. The code also suggests the spectral values for the values of maximum ground acceleration.

The following calculations are based on the IS code. The code used is the IS:1893-1975.

Since the earthquakes occur without any warning, it is very necessary to avoid construction practices that cause sudden failure or brittle failure. The current philosophy relies heavily on the action of members to absorb all the vibrational energy resulting from strong ground motion by designing the member to behave in a ductile manner. In this manner even if an earthquake occurs that is stronger than that which has been foreseen, total collapse of the building can be avoided.

Earthquake resistant designs are generally performed by pseudo-static analysis, the earthquake loads on the foundations are considered as static loads and hence capable of producing settlement as dead loads. Therefore as the footings are generally designed for equal stresses under them, the footings for exterior columns will have to be made wider. Permissible increase in safe bearing pressure will have to depend in the soil-foundation system. Where small settlements are likely to occur larger increase can be allowed and vice versa.

3.2.1 Design seismic coefficients for different zones

The force attracted by any structure during an earthquake is dynamic in nature and is a function of the ground motion and the properties of the structure itself. the dominant effect is equivalent to a horizontal force varying over the height of the structure. Therefore the assumption of a uniform force to be applied along one axis at a time is an oversimplification which can be justified for reasons of saving effort in dynamic analysis. However a large number of structures designed on this basis have withstood earthquake shocks in the past. This is a justification of a uniform seismic coefficient in seismic design. In the code, therefore, it is considered adequate to provide uniform seismic coefficients to ordinary structures.

The IS code suggests two methods for the purpose of evaluation of the earthquake loads. This is similar to the two methods suggested for the calculation of across-wind loads. Both methods calculate the design value of the horizontal coefficient.

Seismic coefficient method

The value of the horizontal seismic design coefficient shall be calculated using the following expression.

$$\alpha_h = \beta I \alpha_0 \quad (3.4)$$

Where

β is a coefficient depending on the soil type. This value varies between 1.0 and 1.5.

I is the importance factor.

α_0 is the basic horizontal seismic coefficient.

The response spectrum method

The response acceleration is first obtained for the natural time period and damping of the structure and the design value of horizontal seismic coefficient is computed using the following expression.

$$\alpha_h = \beta I F_0 \frac{S_a}{g} \quad (3.5)$$

Here

F_0 is a seismic zone factor.

S_a/g is the average acceleration coefficient depending on the natural period and damping of the structure.

3.3 Calculations for a typical case

The calculation of the earthquake load for a typical chimney is given below. The assumptions made are also specified.

The weight data for the case has been taken from the STRAP model of the chimney.

Period of vibration

Diameter of the base = 22.72 m

Base Thickness = 0.649m

Inner diameter at the base is 21.422m

Area of cross section at the base is

$$A = \frac{\pi}{4} (d_{out}^2 - d_{in}^2) \quad (3.6)$$

$$A = 45.0 \text{ m}^2$$

The moment of inertia at the base is calculated by

$$I = \frac{\pi}{64} (d_{out}^4 - d_{in}^4) \quad (3.7)$$

$$\text{The value of } I = 2742.5 \text{ m}^4$$

Radius of gyration r is given by

$$r = \sqrt{\frac{I}{A}} \quad (3.8)$$

$$r = 7.806$$

Hence the slenderness ratio l/r is given by

$$\frac{l}{r} = 32.02 \quad (3.9)$$

The coefficient C_T

$$C_T = 57.822 \quad (3.10)$$

Weight of the chimney

$$W_t = \pi D_{mean} T H_t \rho \quad (3.11)$$

$$\text{Weight} = 17495583 \text{ kg}$$

The period of vibration is now given by

$$T = C_T \sqrt{\frac{W_t h'}{EA g}} \quad (3.12)$$

Substituting the values the value of $T = 125.6$

Design seismic coefficient

Using the Response Spectrum method and the equation **

$$a_h = 0.03975$$

the value assumed are

$$\beta = 1.0 \text{ (assuming a hard/medium soils)}$$

$$I = 1.0 \text{ (importance factor)}$$

$F_0 = 0.25$ (assuming the chimney to be in the zone IV)

Shear force and Bending moments

The design shear force at a distance of X' from the top is given by

$$V = C_V \alpha_h W_t \left[\frac{5}{3} \left(\frac{X'}{h'} \right) - \frac{2}{3} \left(\frac{X'}{h'} \right)^2 \right] \quad (3.13)$$

Where the value of C_V has been found to be 0.2 for the very large time period obtained. Varying the value of X' from 0 to 250 the profile of the shear force has been calculated.

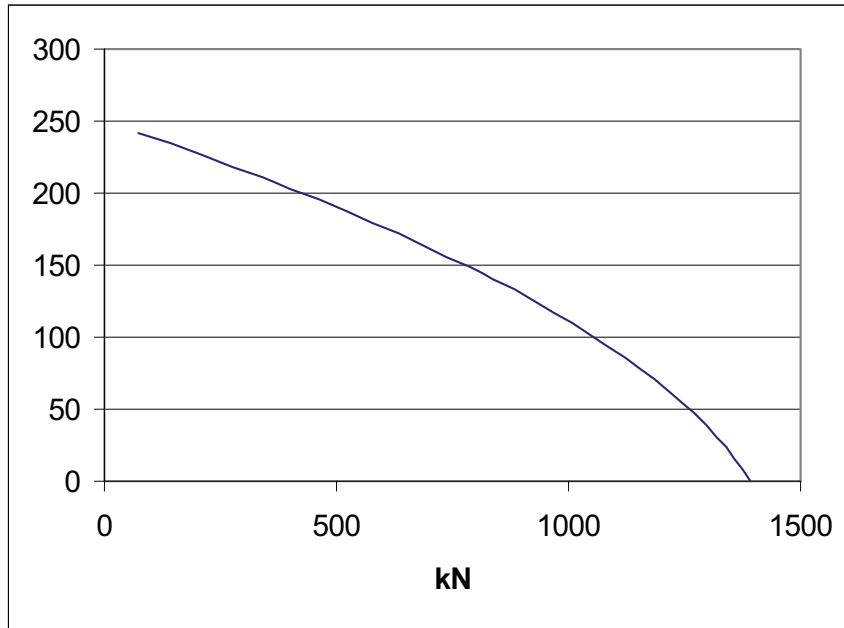


Figure 3.1 – Shear force due to seismic loads

The bending moment can be calculated using the formula

$$M = \alpha_h W_t \bar{h} \left[0.6 \left(\frac{X'}{h'} \right)^{\frac{1}{2}} - 0.4 \left(\frac{X'}{h'} \right)^4 \right] \quad (3.14)$$

Again the value of X' is varied and the expression evaluated. The resultant graph is given below.

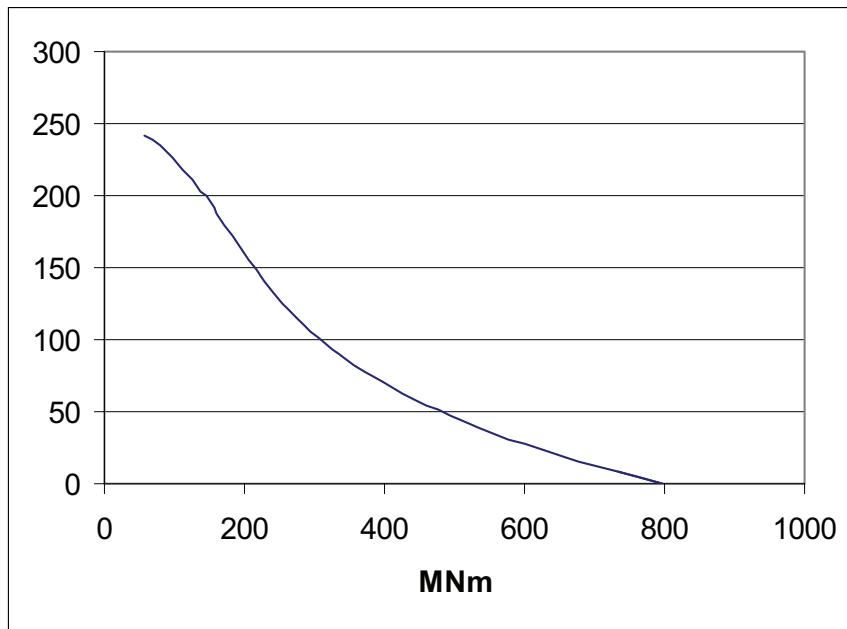


Figure 3.2 – Bending Moment due to seismic loads

As can be seen from the graph, the maximum moment at the base of the chimney is about 800 MNm.

3.4 Conclusions

The reasons and assumptions involved in the evaluation of earthquake loads have been studied. The codal provisions for the calculation of the same have been understood. A sample calculation has been done to calculate the shear force and bending moment caused due to earthquake loading on chimneys. The loads in this case have been found to be significantly lower than those obtained in the wind analysis. Hence earthquake loads do not normally form the main loads to be considered for design.