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SEISMIC QUALIFICATION OF TURBINE BUILDING

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1.0 ABSTRACT

A Turbine building structure, located very close to nuclear safety related structure, is qualified for SSE. Seismic design criteria is developed using Newmark inelastic design response spectra technique. Base shears and strain energies for first mode and for total of 100 modes are compared to ascertain psuedo-SDOF behaviour. Ductility factor and ductile detailing measures are discussed.

2.0 INTRODUCTION

The special functional and safety requirements of safety related concrete structures of a Nuclear Power Plant (NPP) call for design criteria different from those of conventional structures. Protection of site personnel, the public and the environment is established and maintained by effective design criteria against radiological hazards. The structures are designed to satisfy requirements of safety for all loading conditions envisaged during the planned life and also after decommissioning of the plant, till the radiological hazard exists. Safe Shutdown Earthquake (SSE) having a return period of one in 10,000 years, forms an important seismic loading condition for nuclear safety related structures.

In the layout of an NPP, Turbine Building is, generally, isolated from the safety related structures and systems as it does not house any safety related item. In such a case, it is designed as per the design criteria for the conventional structures. However, when the Turbine Building is located very close to any nuclear safety related facility in the layout, it is required to demonstrate that the turbine building will not collapse on the adjacent safety related facility in the event of SSE.

The protection of nuclear safety related facility is either achieved by shifting the Turbine Building sufficiently away from the facility or by designing the Turbine building for SSE load effects which are about 6 to 8 times as high as compared to the seismic design loads for conventional structures. This paper outlines another approach proposing the seismic design criteria which would safeguard against the collapse of such a structure in the event of SSE.

3.0 EARTHQUAKE RESISTANT DESIGN PHILOSOPHY

The load effects due to an earthquake are impulsive in nature. They are not only dependent of the intensity of the design earthquake but also on the dynamic characteristics of the structure. The stiffer the structure, the lesser is its time period and the higher are the design forces in the structure. On the other hand, for a flexible structure, the design forces due to an earthquake are lower. Structures are capable of absorbing and dissipating a considerable amount of energy due to seismic loading, when strained in inelastic region beyond their elastic limit. The energy absorption obtained in the linear elastic portion is only a small fraction of the total energy absorption capability of the structure.

In practice also, such a behaviour is observed in the event of occurrence of an earthquake. Flexible structures have withstood large intensity of earthquakes with lesser damage as compared with stiffer structures.

Considering these aspects, the codes and standards for seismic design specify design intensity of an earthquake as a fraction, say about 1/4 th or 1/5 th, of the total predicted intensity of the earthquake. Special design requirements are also specified in the codes to enhance the ductility of the structure, by virtue of which the seismic energy is dissipated in inelastic deformation of the structure without collapse.

4.0 INELASTIC SEISMIC ANALYSIS APPROACH.

Seismic analysis of an inelastic system is generally performed by step-by-step integration of equations of motion for a given time history considering the material non-linearity. However, it requires very large computer resources. Response spectrum method, on the other hand, is less expensive as small computational resources are required. This method applied to inelastic systems involves development of Inelastic Design Response Spectrum(IDRS), choice of appropriate ductility factor and corresponding ductile detailing measures. The limitation of IDRS Method is that it is valid only for single Degree of Freedom (SDOF) systems, where there is no need of mode superposition. It is observed that the major response of a building structure comes from the fundamental mode. In such cases the structure can be considered as a pseudo-SDOF system for inelastic response spectrum analysis.

The IDRS is developed based on the ductility factor 'μ' representing inelastic energy absorption capacity. For SDOF systems, it is defined as the ratio of the maximum deformation 'δ_m' to the yield deformation 'δ_y'

$$\mu = \delta_m / \delta_y \qquad \text{-------(1)}$$

Newmark and Hall (NUREG/CR-0098) have recommended in their report to USNRC that IDRS can be developed from tripartite elastic spectra by dividing the constant displacement and velocity region by the ductility factor μ, by dividing the constant acceleration region by $\sqrt{(2\mu-1)}$ and retaining the portion beyond zero period acceleration the same.

5.0 DUCTILITY FACTOR

Coats (NUREG/CR-1161/RD) in the report to USNRC has recommended the Newmark inelastic response spectrum technique based on system ductility factor 'μ'

in place of inelastic time history analyses, so long as the total inelastic response is low. A system ductility factor between 2 and 3 is recommended for a class II structure which must not be permitted to cause damage to safety related items housed in it by excessive deformation of the structure. All other structures which are usually governed by ordinary seismic design codes, are in class III with a ductility factor varying from 3 to 8, depending on material, type of construction, etc. UBC 1988 specifies various values of "R_W" factor ranging from 4 to 12 for various structuralsystems .These correspond to the system ductility factors from 1.5 to 4.5.

6.0 APPLICATION TO TURBINE BUILDING

6.1 Building description

The Turbine Building has two bays viz. the Turbo Generator (TG) bay and the electrical bay. The TG bay is 35m wide and 51.6m long with 125 t EOT crane. The electrical bay including passage is 15m wide and 63.2m long. The roof for the TG bay is at EL144.40 m with Ground floor at El 100.0m TG bay has 3 main floors at 107.0m, EL 112.5m, EL120.0m. The electrical bay has 7 floors at EL 107.0 m, 112.5 m, 116.5 m, EL 120.0 m EL 124.5 m, EL 131.0 m, and EL 136.5 m.

The main structural system consists of RC frames arranged orthogonally in plan parallel to each other. T.G. bay has a roof truss in structural steel with necessary tie bracing and purlins. Steel trusses are connected to the RC column to ensure frame action. All other internal floors in TG bay are supported by structural steel framing with cast-in-situ RC slab. All floors in the electrical bay are supported over transverse and longitudinal RC beams. All columns are supported on isolated footings. Typical transverse and longitudinal frames and floor plan of the TB are shown in figure 1,2 and 3 respectively.

6.2 Mathematical Model

The analytical model for the structural system to evaluate 3-D response is prepared in sufficient detail to represent realistically the stiffness and mass properties of the building. The beams and columns are represented by 3-D beam elements. All columns are assumed to be fixed at the top of foundations. The increase in the beam stiffnesses due to finite column widths is considered in the model. The steel beams are modelled to have hinged connection at the beam-column junction. In-plane stiffness of floor slabs is modelled by 3 and 4 noded plane stress elements. Roof truss members & bracings are modelled as truss elements. Elastic modulus of concrete and steel is taken as 28,500 MPa and 200000 MPa respectively and the Poisson's Ratio as 0.2 and 0.3 respectively. Lumping of the mass is carried out at discrete nodal points. Total mass of TB represents a weight of 36400 tons.

6.3 Dynamic Characteristic of T.B.

In order to judge the predominance of the first mode, the free and forced vibration characteristics of T.B. are examined. Analysis is carried out by Subspace Iteration technique to extract lowest 100 modes. To assess forced response characteristics, analysis by response spectrum method is carried out for SSE with 7% damping ratio. The response in various modes is combined using Ten Percent Method of combination. Additionally, building behaviour is investigated for modal strain energies computed as follows.

$$SE_{i,j} = 1/2 \frac{(P_{i,j} * Sa_{i,j})^2}{W_i^{\,2}} \qquad \text{-------(2)}$$

Where

$P_{i,j}$ = Modal participation factor for i^{th} mode in the j^{th} direction

$Sa_{i,j}$ = Spectral acceleration at i^{th} frequency for the j^{th} direction.

W_i = i^{th} modal frequency (rad/sec).

The base shears and strain energies as forced response parameters are presented for the predominant mode in each direction along with those from 100 mode analysis in Table I.

The results in Table I indicate predominance of single mode of vibration in each horizontal direction. It is evident from the table that most of the building response is contained in the fundamental mode in each direction. This establishes pseudo-SDOF behaviour as the basis for application of IDRS method for check analysis of T.B. under SSE.

6.4 Ductility Considerations

TB does not house any safety related equipment. However, its own deformations are to be restricted as it is isolated from adjacent safety related structures by an expansion gap. A ductility factor of 2.5 is chosen for this building. Considering the fact that the structure is to be analysed elastically and designed for the 40 % of SSE load effects , the remaining seismic energy is required to be absorbed in inelastic deformation. The structure should have sufficient ductility so that it deforms beyond elastic limit without collapse. The present seismic design criteria for nuclear safety related structures as per ACI-349 comprising of SSE and OBE analyses, if compared with Newmark inelastic response spectrum technique would lead to a ductility factor of about 1.7. For this ductility requirement, ACI-349 does not recommend any special ductile detailing provision. The level of ductility required for a given structure depends on the quantitative relationship between earthquake intensity and structural strength. Given a design intensity or a design response spectrum indexed by an effective peak acceleration , it appears plausible to relax some of the detail requirements, if the design strength is increased with respect to the minimum codal requirement. The structure as per IS codes falls under zone III corresponding to requirements of detailing for high seismic risk zone. The chosen ductility factor here is lower than the ductility factors inbuilt in the codal provisions. Hence the design earthquake intensity is higher than that specified by codes. Thus detailing requirement corresponding to moderate seismic risk zone are considered to be appropriate.

6.5 Inelastic Design Response Spectrum.

Inelastic design response spectrum for a system ductility factor of 2.5 is developed from the 7% SSE design spectra using the method recommended by Newmark and Hall. Elastic and Inelastic design response spectra for 7% damping ratio are shown in Figure 4.

6.6 Design Check

The structure is subjected to other relevant load conditions in addition to earthquake such as dead loads, live loads, conductor tension, crane load, etc. The RC members and structural steel members are checked for safety from strength consideration as per IS:456 and IS:800 respectively. Any deficiencies in the provision are made up adequately to ensure safety. In line with the ductility requirements of ACI-318(89) for moderate seismic risk zone, web reinforcement is provided considering twice the value of shear due to earthquake effects. In order to avoid brittle failure, the load effects due to earthquake are doubled in the design of truss connections to the RC columns.

7.0 CONCLUSION

Seismic design criteria for non-nuclear safety related structure located very close to a safety related structure are established using Inelastic Design Response Spectra. A turbine building structure, chosen as a case study, is reviewed for its free and forced response characteristics to establish its psuedo-SDOF behaviour. Base shear and strain energy are found to be a good response parameters to judge the above aspect. IDRS is developed using ductility factor of 2.5. With appropriate ductility provisions, TB is qualified for SSE.

8.0 ACKNOWLEDGEMENTS

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TABLE –I
FREE AND FORCED RESPONSE CHARACTERISTICS OF TURBINE BUILDING

1.	DIRECTION	TRANSVERSE	LONGITUDINAL
2.	PREDOMINANT MODE	1	2
3.	FREQUENCY	0.64 Hz	0.73 Hz
4.	MODAL MASS IN PREDOMINANT MODE	67.14%	84.66%
5.	TOTAL MODAL MASS IN 100 MODES	99.40	99.60
6.	BASE SHEAR IN PREDOMINANT MODE	4130.20 T	5446.00 T
7.	TOTAL BASE SHEAR IN 100 MODES	4883.30 T	5587.00 T
8.	RATIO (6)/(7)	0.85	0.98
9.	MODAL STRAIN ENERGY IN PREDOMINANT MODE	208.00 TM	244.70 TM
10.	TOTAL MODAL STRAIN ENERGY IN 100 MODES	211.00 TM	245.10 TM
11.	RATIO (9)/(10)	0.99	0.99

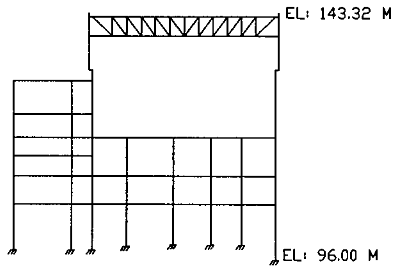

FIG 1 : TRANSVERSE FRAME

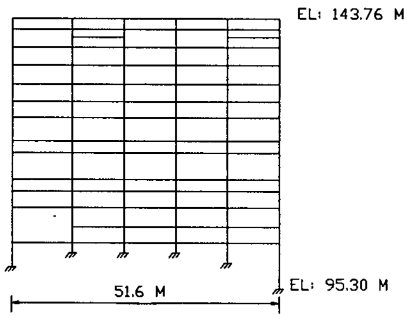

FIG. 2 : LONGITUDINAL FRAME

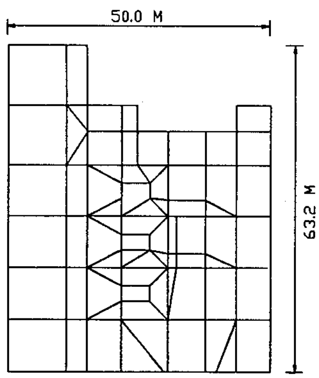

FIG. 3 : FLOOR PLAN

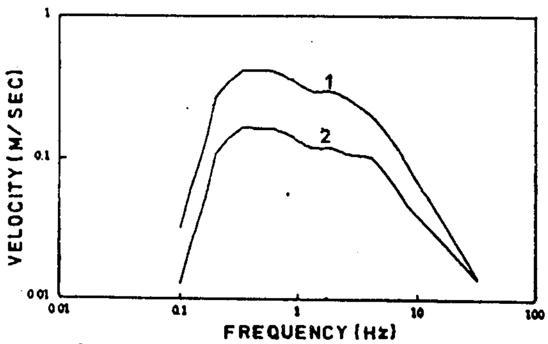

FIG.4 : 7 % DAMPING SSE GROUND SPECTRA