V° WCEE: Aseismic Design of Nuclear Power Plant

On Design Criteria for Equipment Mounted on a Massive Structure

by Alberto Castellani *

S U M M A R Y

The design earthquake response spectrum for the site is scarcely sufficient as data for an analysis of the seismic response of appendices or equipment, maching safety requirements to reasonable software. The upper bound frequency for the meaningful Fourier components of ground acceleration, and the earthquake duration should also be known. In particular, if the set of expected earthquake accelerograms is defined for the site and the equipment motion is coherently evaluated, the strength requirements for appendices or equipment may be substantially reduced.

In spite of this reduction, a noticeable conservatism in again intrinsically present when the maximum acceleration at the point of support is evaluated by modal analysis. In fact upper stories accelerations are overstimated.

I N T R O D U C T I O N

There is growing interest in the safety analysis of structural appendices and of equipment for industrial buildings. As to dynamical analysis, there may be remarkable amplification in the motion of the supports. As to economy and security the same attention as for the main structure may be required. On the other hand their dynamic analysis could be more costly and time consuming. This is the case for equipment in a nuclear power station. In a typical BWR reactor, for instance, the total length of piping may reach some tens of Km with an equivalent cost of 15-20 dollars per meter for their dynamic analysis.

The analysis usually requires the definition of a floor response spectrum for equipment of the same meaning as a ground response spectrum for structures. Feedback is generally neglected between the equipment motion and that of the structure, for light equipment.

* Assistant Professor, Politecnico di Milano, P.zza Leonardo Da Vinci, 32 - Milan, 20133 Italy.

Heavy equipment is incorporated in the dynamical model of the entire structure.

According to widely accepted practice, a design earthquake is assumed for the site with a smoothed response spectrum. This generally represents a spectrum of the required strength instead of a typical local earthquake spectrum. In fact the selected site is generally not in an active zone so local strong motion records are not available. Floor response spectra are then evaluated by taking into account the amplification of the equipment/structure motion in some earthquakes as a function of the dynamic behaviour of both equipment and structure. This amplification is again smoothed for the proposed calculation of floor response spectra.

Notwithstanding the two smoothing procedures, the floor response spectra so evaluated may show pronounced peaks. Since the structure is to the equipment what the local subsoil is to the structure, then logically peaks may be present. In fact if precise knowledge of the dynamic behaviour of the local soil were available, the ground response spectrum could also be defined with suitable peaks.

The application of this method seems open to argument.

1) In some applications, the sharpness and the height of the peaks is the result of deducing amplification curves from peaked earthquakes combined with the hypotesis of having the main peak for each of the natural periods of the structure. Such an hypothesis is not coherent either with the risks philosophy or with the seismic analysis of the main structure itself, which is based on a smoothed ground response spectrum.

2) At some points of a structure the application of the above method might lead to acceleration 20 times that of the ground for equipment with 1% damping. A time-history-integration rarely leads to such an amplification.

3) A sharp-peaked floor response spectrum requires a precise dynamic analysis for the equipment. On the other hand some of the equipment in any one structural system may be designed by different teams or at different times: (e.g. a pump and the piping system). So the complete knowledge of the dynamic model of the system, necessary for the application of the above method, would require a proper modification in time schedules.

4) Quite often a considerable proportion of the equipment is too complex for the available dynamic codes, but when properly analyzed on a shaking table, they show rigidity or only one degree of freedom. An alternative static verification through suitable horizontal forces might be advisable.
5) On the other hand a piping system may have so many natural frequencies close to each other, that the uncoupling of their natural modes, and the classical mode superposition may be questioned.
6) In most cases, mechanical engineers analyze the dynamic behaviour of equipment or piping by adapting the static stress programs available, and the simplest way to do this is to use static equivalent forces.

It may be further mentioned that respecting the time schedule often requires the definition of input for equipment when some of the soil or of the building characteristics are not yet completely known. This implies taking into account several possible configurations, and enveloping the worst one: thus the escalation of the safety factor, if coherently applied, would produce unacceptable software. For the BWR ENEL IV at Caorso, for instance, several tens of configurations were run for the reactor building. When the method [4] is applied, the aspect of the floor response spectrum at one point of the reactor is as in Fig.2. The most severe condition is not always that coinciding with the maximum acceleration of the point of support.

Some of the above difficulties would be reduced if part of the dynamic analysis could be shown to be superfluous and if a simplifying smooth floor response spectrum could be used.
This might well be done without recourse to the envelopes of the peaks.

An ideal but expensive procedure for evaluating floor response spectra would be: select some representative earthquake records for the site, and derive time histories of the floor movements. This ought to be repeated to cover the uncertainties of soil and structural behaviour.

When such a procedure is abandoned for a more tested modal analysis, some conservatism is necessarily introduced, as will be shown. But it is possible to avoid an escalation of useless conservatism if during the seismological evaluation of the site, two further items are defined:

1) An appropriate upper bound for the meaningful Fourier components of the accelerograms for the site. Frequencies of 33 cps, suggested by IEE, [11] or 30 cps, suggested by Small are [16] certainly upper frequencies but are too conservative for a station on soft soil or when the design earthquake is expected from an active source but several miles away. All equipment whose natural frequency is obviously higher than that tie-frequency can be considered rigid.

2) The origin of the set of expected earthquakes for the site. If even small magnitude earthquakes are expected in the vicinity they may have high acceleration, but short duration, thus providing softer amplification curves for the evaluation of floor response spectra. For the sake of coherence in safety analysis, only the expected records for that site are to be used for finding the average amplification curves in the range of periods of interest.
This approach reduces the peak in the amplification curve, see Fig.3.

2) CALCULATION OF THE FLOOR RESPONSE SPECTRUM.

The earthquake response of appendices or equipment has been accounted for through three techniques: 1) Time-history of the equipment support motion; 2) stochastic approach, which represents the input motion by the power density of the expected ground motion accelerograms; 3) modal analysis applied both to the main structure and to the equipment.

The advantages and disadvantages of each of the three approaches are well known. For instance, the time-history analysis seems particularly suited to piping systems. In fact a piping system is often supported at different levels, so the response spectrum at any particular floor, loosing any phase information, is no longer suitable for defining the motion of the supports. Besides this piping system may have so many natural frequencies close to each other, that the uncoupling of their natural modes and the usual mode superposition might be unjustifiable. On the other hand the time-history technique requires normalization of the maximum acceleration of the ground and there is a strong tendency to select the normalization factor so that its response spectrum will everywhere envelope the specified site response spectrum. Such a tendency produces, for the general case, higher safety factors than those involved within the specification of the site response spectrum.

Besides this if more building configurations and more earthquake records are to be taken into account, unacceptable software will be produced.

So none of the three methods can be given preference. Nevertheless, the arguments here proposed apply to any one of then. For the sake of brevity only modal analysis of both building and appendices will be discussed in the present paper. It may also be said that the stochastic approach can be so flexible as to take into account the main items involved, for instance, the frequency content of ground motion or the duration of the earthquake [1,13].

The modal analysis approach for the building gives at the point in question the maximum accelerations ${}^{n}\ddot{S}(P_o)$ due to each mode. Each one of these contributions ${}^{n}\ddot{S}(P_o)$ is partly due to the amplification of the ground motion at the mode frequency ω_n, and partly due to the mode relevance at the point P_o. The well known formula is:

$$ {}^{n}\ddot{S}(P_o) = \omega_n \, S_v(\omega_n) \, {}^{n}\phi(P_o) \, g_n \qquad \text{---- 1} $$

where $S_v(\omega_n)$ is the velocity response spectrum at ω_n;
${}^{n}\phi(P_o)$ is the mode amplitude at P_o;
g_n is the modal partecipation factor.

Now consider an oscillator with frequency ω_n supporting elastically a light mass, see Fig.1. When the base of the oscillator is acted on by an earthquake motion, compute the maximum acceleration of m_2.

Biggs and Roesset have shown a reasonably approximate procedure by which the ratio A between the acceleration of m_2 and that of m_1 may be represented as a function of ν_1, ν_2 the selected earthquake, and of the ratio ω_1/ω_2:

$$ A = A\left(\nu_1, \nu_2, \omega_1/\omega_2\right) \qquad \text{-------- 2} $$

(for the range of validity of Eq.2 the reader is referred to [4]). So the acceleration of m_2, when the point P_o is moving in the mode n, is:

$$ {}^{n}a_2 = \omega_n \, S_v(\omega_n) \, {}^{n}\phi(P_o) \, g_n \qquad \text{--- 3} $$

The author maintains that this amplification A is to be selected depending on the characteristics of the set of earthquake records suitable for the site.

In Mediterranean seismic zones, for instance, hight accelerations have been recorded with small magnitudes and short durations. This is the case for the Ancona earthquakes of '72, when in the epicentral zones accelerations of up to 0,6 g were recorded. The six more violent accelerograms were used for the sistem in Fig.1, for the range of periods 0.33$\div$ 2 sec. The average amplification curve is sensibly smaller than that suggested by longer earthquakes, as Fig.3 shows.

3) CONSERVATISM OF THE MODAL ANALYSIS APPROACH

As far as a one degree of freedom oscillator is concerned, the maximum displacement of its mass relative to the ground is:

$$s = \frac{S_V(\omega_0)}{\omega_0}, \qquad \text{------ 4}$$

and the equivalent static force F satisfies:

$$\frac{F}{m} = \omega_0 S_V(\omega_0) = \omega_0^2 s, \qquad \text{------- 5}$$

where S_V is the velocity response spectrum and m the mass of the oscillator.

The last quantity $\frac{F}{m}$,may approximatively represent the absolute acceleration for the mass of the oscillator.

As to a multi-degree-of-freedom system, the theory of modal analysis requires computing the relative displacement vectors $\{^n s\}$ and then the equivalent static forces through the product of stiffness matrix $[K]$ by the vectors $\{^n s\}$:

$$\{^n F\} = [K]\{^n s\}, \qquad \text{------6}$$

n being the mode index. The last quantity may also be expressed as:

$$\{^n F\} = \omega_n^2 [M] \{^n s\}, \qquad \text{------7}$$

where ω_n is the mode frequency. This expression suggests representing the absolute acceleration of each mode through the vector:

$$\{^n a\} = \omega_n^2 \{^n s\}. \qquad \text{---------8}$$

It is easily shown that by this procedure the pseudo-acceleration ascribed to lower stories tends to zero instead of to the ground acceleration value, (larger errors would arise if quantities in Eq.8 were to be considered as the relative acceleration to be combined with ground acceleration).It is exactly zero when no ground compliance is taken into account.

On the other hand this procedure overestimates the absolute accelerations in upper stories. So while in lower stories the underestimated accelerations may be reasonably adjusted-at least when they are less than the ground acceleration-no satisfactory procedure is available for correcting the upper stories,to the author's knowledge.

Equipment accelerations are also overestimated because coupling between structure and equipment motion is necessarily neglected.

Now refer to the method of [4] .By an application of modal analysis both to equipment and structure, the motion of the quipment is represented by a suitable combination of the motions of the mass m_2 in a set of elementary models of the type in Fig.1. Here m_2 is the modal mass of the equipment and m_1 that of the structure.
Cases in which $m_2 \ll m_1$ are considered.

Let the natural frequency $\sqrt{K_2/m_2}$ of the attached mass be equal to the frequency of the main mass, $\sqrt{K_1/m_1}$ .When the forcing function is $\ddot{y}\cos(\omega t)$ where ω is the common frequency, the feedback from the equipment provides an effective mechanism of energy dissipation, and in fact the Frahm's dynamic vibration absorber (1909) was idealized by such a model.

When an earthquake ground motion is acting on the system of Fig.1, the energy dissipation due to feedback from m_2 to m_1 is no longer as effective as in Frahm's absorber. Nevertheless the amplification curves are again reduced. Fig.4 shows this reduction in amplification curves. The curves are obtained as average amplifications in the set of periods 0.25$\div$1 and for the Taft and El Centro accelerograms.

In order to appreciate numerically the influence of this coupling effect, the mass ratio m_2/m_1 will now be analysed.It is the ratio between the modal mass of equipment to that of the structure. Equipment may often be configurated as a mass on a massless support [15] so the value of m_2 is the mass of the equipment itself. On the other hand the modal mass m_1 of the structure may be much smaller than the mass of the entire structure: for instance, when in the considered mode of vibration the foundation is fixed and only a portion of the structure is deformed. In the general case, it can be shown that the effective mass to be accounted for is:

$$ {}^{i}m_1 = \frac{\int_S \rho(P)\,[{}^{i}\varphi^2(P)]\,dV}{{}^{i}\varphi^2(P_o)} \tag{9} $$

where: $\rho(P)\,dV$ is the mass of the structure in the elementary volume dV around the generic point P ;

$\varphi(P)$ is the mode shape in the mode i ;

P_o is the point where the floor response spectrum is evaluated.

Two comments on Eq.9 :1) when higher modes are involved ${}^{i}m_1$ is only a small portion of the entire mass, so the mass ratio m_2/m_1 may be significant; 2) a time-history analysis on a sophisticated structural model, may show acceleration peaks due to local deformability of the structure.
If the equipment coupling is not accounted for, such local peaks are no longer reliable for the purpose of floor response spectrum evaluation.

4) AKNOWLEDGMENTS

The Author is deeply endebted to E. Iansiti, Head of the Security Division of the Italian Committee for Nuclear Energy for constantly promoting the development of structural analysis in this field.

5) REFERENCES

1) AMIN, A., ANG, A.S., "Statistical Treatment of Seismic Loading on Reactor Buildings and Equipments" I° Conf. on Struct.Mech.in Reactor Technology, Berlin, 1971.

2) AMIN, M., HAll, H.S., NEWMARK N. and KASSAWARA, R.F.,: "Earthquake Response of Multiply Connected Light Secondary Systems by Spectrum Methods", I° National Congress of Pressure Vessel and Piping Technology, ASME, San Francisco, Cal. May 1971.

3) "An Interpretive Review of Seismic Design Methods" ORNL TM-2900 Oak Ridge, Tenn:Oak Ridge National Laboratories, May, 1970.

4) BIGGS, J.M. ROESSET, S.M., "Seismic Analysis of Equipment Mounted on a Massive Structure", Seismic Design for Nuclear Power Plants" Edited by Hansen R.J. - M.I.T. Press, 1970.

5) CIMORELLI, L., FEDERICO, A., MARENCO, G., MUZZI, F., ZAFFIRO, C.,: "Analisi teorica e sperimentale delle vibrazioni di una tubazione spaziale," Convegno Nucleare di Pisa, Sept.1970.

6) GARRO, A., MALAMBRI, P., ROBERTI, R.,: "Analisi dinamica di tubazioni" Convegno Nucleare di Pisa, 1970.

7) HARRINGTON, R.L., VORNS, W.S.,: "Dynamic Shock Analysis of Shipboard Equipment", Meeting of the Society of Naval Architectures and Marine Engineers, Hampton Roads, 1966.

8) HISADA,T., et Al: "Philosophy and Practice of the Aseismic Design of Nuclear Power Plants:Summary of the Guidelines in Japan" I° Conf. on Struct.Mech. in Reactor Technology, Berlin, 1971.

9) HURTY, W.C.,: "Dynamic Analysis of Structural Systems Using Component Modes" AIAA, April 1965.

10) IANSITI, E.,:"Nuclear Safety Considerations Concerning the Construction of a Nuclear Power Plant in Seismic Zones, "IAEA Meeting on Aseismic Design and Testing of Nuclear Facilities, Tokyo, June '67.

11) IEE Standard: "Criteria for Protection Systems for Nuclear Power Generating Stations" IEEE 279 New York N.Y.: IEEE Publications Department. 1971.

12) "Interaction of a Seismic Disturbance with a Reactor Facility "ORNL, TM 2901, Oak Ridge, Tenn.: Oak Ridge National Laboratories, May, 1970.

13) LAZZERI, L.,: "Computation of the Floor Response Spectra by the Use of a White Noise Technique", Work still in progress, discussed in the Meeting Pisa, October 1972.

14) LEVY,S.,:"G. Limiting Attachement for Equipment,"ASME Conference Philadelphia,Pa.,March 1969.6S. VIBR-23.

15) LIN, W.C.,:"Seismic Analysis of Equipment in Nuclear Power Plants", I° Conf.on Struct.Mech. in Reactor Technology, Berlin 1971.

16) SMALL, R.,:"Current Considerations in the Seismic Design of Nuclear Power Station Equipment", IEEE Winter Meeting, New York, N.Y. 1972.

17) STOYKOVICH, M.:"Variation of Input Parameters Considered in Determining Floor Response Spectra" *GIBBS and Hill, New York, N.Y. 1972.*

18) "Title 10 Part 50, General Design Criteria for Nuclear Power Plants" Federal Register, Vol.36 pp. 3255-3200, Febraury 20, 1971. Washington, D.C.: United States Government Printing Office.

19) U.S.A.E.C.: "Summary of Current Seismic Design Practice for Nuclear Reactor Facilities",TID 25021, Sept.1971.

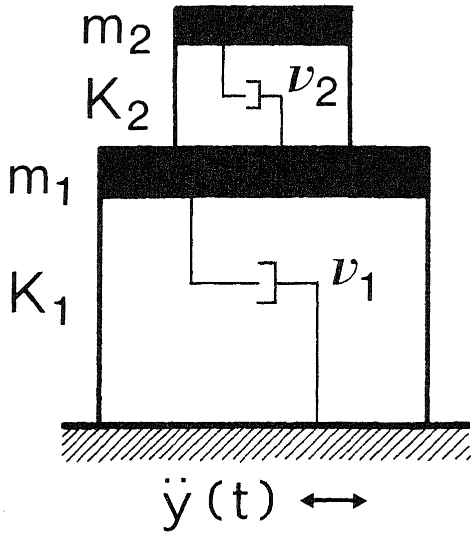

Fig.1

The elementary model obtained by applying modal analysis independently to the building and to equipment.

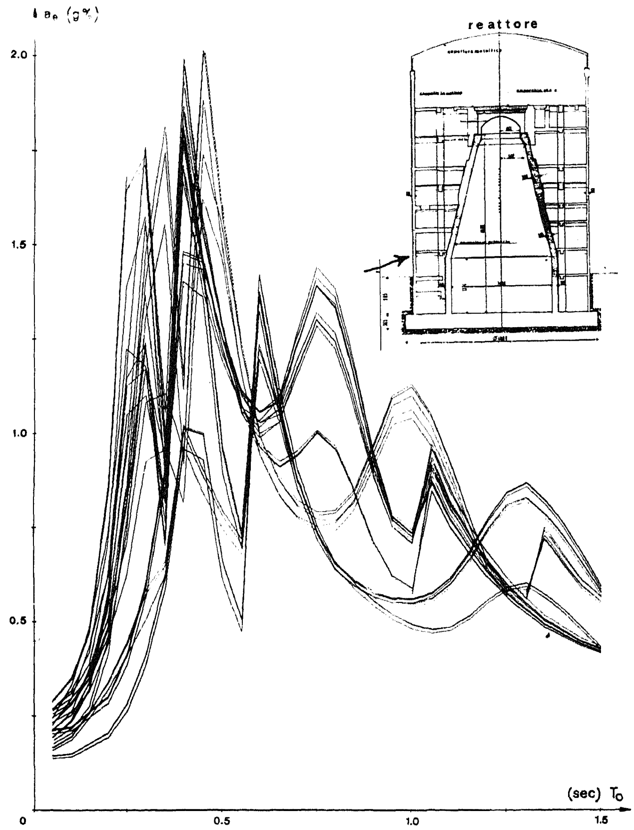

Fig. 2: Example of floor response spectra at a point of a reactor building. Two different soil conditions two different damping distribution, and 9 different models for the structure are considered. Ground acceleration .2 g.

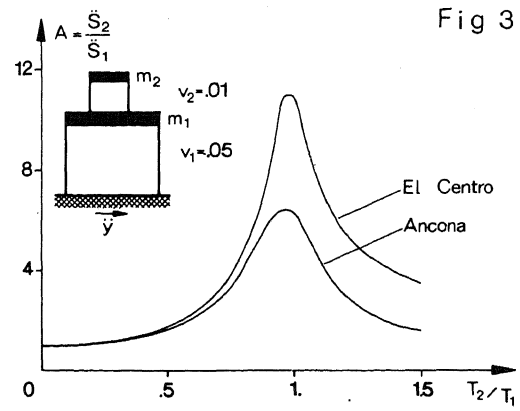

Fig 3

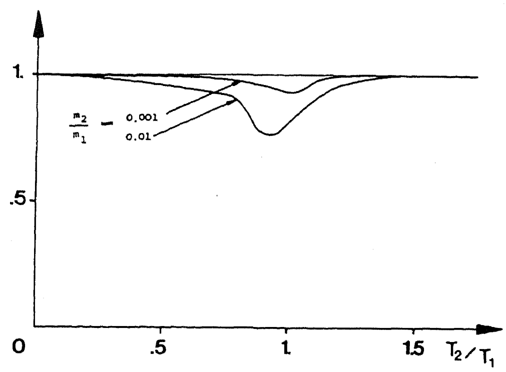

Fig.4 Reduction for $A = \ddot{S}_2/\ddot{S}_1$ due to feedback from m_2 to m_1 .