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DESIGN SPECTRA FOR THE NEW GENERATION OF CODES

Peter FAJFAR

University of Ljubljana, Faculty of Civil Engineering and Geodesy
Jamova 2, 61000 Ljubljana, Slovenia

ABSTRACT

In the paper, inelastic spectra for strength, displacement, hysteretic and input energy, based on constant ductility, are discussed. General recommendations for design spectra to be used in the future seismic codes are given.

KEYWORDS:

Inelastic design spectrum; response spectrum; displacement spectrum; energy spectrum; seismic codes; reduction factor; ductility factor; earthquake resistant design; energy dissipation; cumulative damage.

INTRODUCTION

In seismic codes, design spectra are used for the determination for seismic demand. Although theoretically limited to the linear elastic behaviour of structures and single-degree-of-freedom (SDOF) inlelastic systems, the spectrum approach can, in many cases, also be applied approximately to the analysis of the inelastic behaviour of multi-degree-of-freedom (MDOF) structures. Elastic spectra depend on the characteristics of the expected ground motion and viscous damping. They represent the basic, but incomplete information on the ground motion. In this paper, elastic spectra are not discussed, they are assumed to be known. Inelastic spectra depend not only on the characteristics of the expected ground motion at the site but generally also on the nonlinear characteristics of the structural system, what severely complicates the problem. In the paper, in addition to the most commonly used acceleration (or strength) spectra, displacement and energy spectra are also discussed. Recommendations for design spectra to be used in future codes are given.

STRENGTH SPECTRA

An inelastic design spectrum is usually synonymus with the strength spectrum which basically represents a reduced elastic (pseudo)acceleration spectrum. It can be obtained by a reduction of an elastic spectrum through the use of reduction (or behaviour) factors, or by direct derivation through statistical studies of the spectra obtained by the nonlinear dynamic analysis of SDOF systems subjected to ground motions (e.g. Lawson, 1995, Riddell, 1995). Both approaches yield results of satisfactory accuracy for practical purposes. Although the direct approach with reduction factors may be less accurate than the direct one. Alternativelly, inelastic response can be estimated by using an equivalent elastic system with increased period and damping and applying elastic design spectra. Only the reduction of elastic spectra will be discussed in this paper, since this indirect approach is the most versatile.

The degree of reduction of design spectra used in codes is based mainly on experience and judgement, and depends primarily on the provided or tolerated ductility and on the overstrength. The reduction factor R, which is defined as the ratio of elastic strength demand mA_e (where m is mass and A_e is the value in elastic (pseudo)acceleration spectrum) to design strength for a specified ductility factor, can be written as (Fischinger and Fajfar, 1990, Uang, 1991)

$$R = R_\mu R_s \tag{1}$$

where is R_μ the ductility dependent reduction factor, defined as the ratio of elastic strength demand to inelastic strength demand F_y, and R_s is the overstrength factor, defined as the ratio of the actual strength (inelastic strength demand) to design strength.

R_μ depends on the period. It monotonically increases from 1 to a value near to μ with increasing period. Miranda and Bertero (1994) made an overview of some recent proposals for R_μ factors, which take into account this feature.

R_μ factors are just a means to obtain inelastic spectra from elastic spectra. They are derived using specific elastic spectra and, in principle, they can be applied only in connection with the same elastic spectra as used in derivation, or with spectra similar to them. This should be noted when comparing R_μ factors obtained by different authors. Smooth R_μ spectra should be used only in combination with smooth elastic spectra. For evaluation of R_μ factors are the most important the final result in terms of strength, displacement and energy spectra. Vidic et al (1994) have demonstrated that quite different R_μ spectra may lead to very similar strength spectra.

The close link-up of R_μ and elastic spectra has to be considered when determining seismic demand in the case of soft soils that has recently attracted a lot of attention (e.g. Miranda, 1993, Rahnama and Krawinkler, 1993, Riddell, 1995). The main feature of a ground motion on a soft soil site is its narrow band frequency content. Typically, sharp peaks can be observed in elastic spectra at the period which coincides with the "predominant" period of the ground motion. However, at the same period peaks in reduction factors R_μ can be observed, as well. So, the resulting inelastic strength spectra are relatively smooth. It should be noted (as discussed e.g. by Tarquis and Roesset, 1992), that common smooth spectra for R_μ factors developed by statistical analyses of ground motions with broad band frequency content are not applicable in combination with narrow band elastic spectra. Vice versa, R_μ factors developed for ground motions with narrow band frequency content should not be used in combination with broad band elastic spectra. Consequently, if the ground motion on a soft soil site and the corresponding elastic spectrum can be reliably predicted, then special site-specific procedures for determination of inelastic strength demand are needed. However, if the available data do not permit a reliable prediction of the "predominant" period of the design ground motion and a broad band elastic spectrum is used (e.g. a design spectrum for soft soil provided in different codes), then general purpose period dependent R_μ factors (like those proposed by Vidic et al, 1994) or smooth R_μ factors developed for soft soils (e.g. Ridell, 1994) can be used for the determination of the required strength.

Strength exceeding that required by codes (overstrength) is a major factor contributing to the seismic resistance of structures. Bertero has frequently pointed out the importance of the overstregth (e.g. Bertero, 1986) and called it "the blessing" which helped buildings to survive major earthquakes. Osteraas and Krawinkler (1989) explained good behaviour of short-period steel moment-frames in Mexico City by the large overstrength. They clearly showed that the overstrength increased significantly for short-period structures. The same conclusion was obtained by Nassar and Krawinkler (1991) for several common types of steel and reinforced concrete structures, and by Fischinger et al (1994) for RC buildings with structural walls designed according to Eurocode 8. Additional evidence on overstrength can be found in a large number of publications dealing with nonlinear behaviour of buildings. The most important sources of overstrength can be approximately quantified by a nonlinear push-over analysis.

Implementation in codes

The code suggested values of R-factors are essentially of an empirical origin. So, in addition to ductility, they generally automatically imply overstrength, although this is usually not explicitly realised. However, using a single number (R factor) to account for the implicit interaction of at least two parameters (ductility and overstrength) my be misleading for a designer. If he attempted to provide a structure with just the minimum strength required by codes, it would be very difficult to ensure enough ductility to a building designed for large (e.g. R = 8) behaviour factor (Bertero, 1986). If lower reduction factors were used, the design would be too conservative in the majority of cases, since the majority of buildings do possess overstrength. An appropriate solution appears to be to define the R factors in aseismic codes according to eq.(1) as the product of the equivalent global ductility factor R_μ and the overstrength factor R_s.

First attempts in this direction have been already made in the Swiss (SIA 160) and the Canadian codes (NBCC 91), where explicit constant factors have been introduced that can be interpreted as overstrength factors (1/0.65 in the Swiss code and 1/0.6 in the Canadian code). In principle, a correct and not excessively demanding way for the determination of the overstrength, which takes into account several important sources of overstrength, is a nonlinear static (push-over) analysis. Such an analysis is required in the draft of new Japanese guidelines for 31 to 60 m high RC buildings (Otani et al, 1994). A simple limit analysis can be used for buildings less than 31 m high. According to Eurocode 8, for a very limited number of cases (some types of steel and composite buildings) the design seismic loading depends on the actual strength of the structure that may be determined by non-linear analysis. Push-over analysis is a part of new seismic design methodologies under development in the U.S.A. (e.g. SEAOC Vision 2000, ATC-33, ATC-34).

DISPLACEMENT

As a consequence of the fact that, in the case of severe earthquakes, displacements are more important than forces, different displacement based design approaches have been proposed. Not enough attention, however, has been paid to displacement (D) spectra. Only few authors have explicitly proposed general purpose D-spectra or general procedures for the determination of inelastic displacements.

The inelastic displacement spectrum can be obtained from the elastic (pseudo)acceleration spectrum by the formulae (Fajfar, 1995b)

$$D = \frac{\mu}{R_\mu}\frac{A_e}{\omega^2} = \frac{\mu}{R_\mu} D_e = \mu\, R_s\, D_d \tag{2}$$

where ω is the natural frequency, D_e is the elastic displacement (spectrum) ($D_e = A_e/\omega^2$) and D_d is elastic displacement (spectrum) due to design loading F_d. If the reduction factor R_μ is approximately equal to the ductility factor μ, the inelastic displacement is approximately equal to the elastic displacement, or vice versa, as assumed in the derivation of the formula $R_\mu = \mu$, based on the "equal-displacement rule". In the short-period range, however, the displacement spectrum is highly influenced by the characteristics of the non-linear behaviour of the structures. The inaccuracies in the R_μ spectrum usually result in an unrealistic displacement spectrum. It seems that approximate R_μ spectra proposed by Vidic et al (1994) are accurate enough to be used for the determination of displacement spectra according to eq. (2).

The displacement demand in code is usually computed as

$$D = R_D\, D_d \tag{3}$$

where the value of the factor R_D is equal to or similar to the value of the reduction factor R. By comparing eqs. (3) and (2) and considering eq. (1) and relation between μ and R_μ it can be seen that eq. (3) underestimates the displacement demand in the short-period region.

In the long-period range, design acceleration spectra are typically conservative in order to take into account influence of higher modes, and/or to accomodate the requirement for minimum seismic loading. This conservatism greatly increases the displacement spectra determined according to eq. (2). So, the basic elastic acceleration spectra, without corrections made in the process of converting smooth response spectra to design spectra, should be used as a base for the determination of displacements. Furthermore, it should be taken into account that the displacements in the very long-period range are limited by the fact that the relative displacement of a very flexible SDOF system is equal to the ground displacement. Unfortunately, the implementation of this requirement is difficult due to uncertainties connected with maximum ground displacements.

ENERGY

Already in the late fifties Housner proposed "a limit design type of analysis to ensure that there was sufficient energy-absorbing capacity to give an adequate factor of safety against collapse in the event of extremely strong ground motion". However, for about a quarter of a century the energy concept was ignored in earthquake-resistant design because of the apparent complexities in the quantification of energy demands and capacities and their implementation in the design process. It is only recently, that the energy concept has attracted more attention in the research community. The input energy E_I is related to the cumulative damage potential of ground motions, and the dissipated hysteretic energy E_H is the structural response parameter which is often correlated to cumulative damage in structures. Energy parameters incorporate the influence of both strength and deformation and they include the effect of duration of strong motion. Maximum input energy is relatively insensitive to structural characteristics. According to Akiyama (1985) the whole E_I design spectrum can, for most practical purposes, be assumed to be independent on structural characteristics. So, structural resistance and earthquake loading can be uncoupled.

For a practical design procedure, which takes into account cumulative damage, general purpose energy spectra are needed. Akiyama (1985) proposed a simple bilinear spectrum for energy based velocity $v_E = \sqrt{2\ E_I/m}$. The bilinear v_E design spectrum can be constructed as an envelope of the elastic v_E-spectrum for 10 per cent damping. Recently, the same author developed a procedure for calculation of site specific spectra (Akiyama and Kitamura, 1992). Alternatively, Kuwamura and Galambos (1989) expressed the maximum value of E_I, which is imparted to systems with their natural periods in the vicinity of the characteristic ("predominant") period of ground motion T_c, as a function of T_c and $\int \ddot{u}_g^2(t)\,dt$, where $\ddot{u}_g(t)$ is ground acceleration. Kuwamura et al (1994) proved that v_E spectrum is, for an elastic system, basically equal to the smoothed Fourier amplitude spectrum.

A basically different approach for determination of energy spectra was developed by Fajfar and Vidic (1994). It is based on the cumulative damage parameter γ described in next subsection. It yields energy spectra which are in short- and long-period regions more realistic than simple bilinear spectra. A similar approach was applied by Nurtug and Sucuoglu (1995).

Hysteretic to input energy ratio E_H/E_I is one of the most stable quantities in earthquake resistant design. Simple formulae for this ratio which depends mainly on the viscous damping, and, to a smaller extent, on ductility and hysteretic behaviour, were proposed (e.g. Akiyama, 1985, Kuwamura and Galambos, 1989, Fajfar and Vidic, 1994). Knowing E_H/E_I spectra, E_H spectrum can be easily obtained from E_I spectrum and vice versa.

In the case of near-field strong ground motions significant acceleration pulses can occur. Anderson and Bertero (1987) noticed that the nonlinear response of structures is particularly sensitive to the pulse duration relative to the fundamental period of the structure and to the pulse acceleration relative to the yield resistance seismic coefficient of the structure. Naeim (1994) showed that, in the case of acceleration pulses, most of the input energy is exerted on the structure in a short time interval and the structure does not have enough time for cyclic vibrations to utilize structural damping efficiently. A large portion of the energy has to be dissipated through E_H, resulting in an increase of the E_H/E_I value.

Cumulative damage parameter γ

The parameter γ controls the reduction of the deformation capacity of structures due to cumulative damage (Fajfar, 1992). It is proportional to the ratio between the equivalent velocity (based on hysteretic energy) and pseudo-velocity. It represents a normalized form of the hysteretic energy and a ratio between the square root of the hysteretic ductility μ_H and displacement ductility μ

$$\gamma = \frac{\sqrt{E_H / m}}{\omega\, D} = \frac{1}{\mu}\sqrt{\frac{E_H}{F_y\, D_y}} = \frac{\sqrt{\mu_H}}{\mu} \tag{4}$$

Parameter γ can be also interpreted as the square root of the number of complete cycles in the elastic SDOF system (when it is forced to displace to the displacement $\pm D$) that are required for absorbing the amount of energy equal to E_H (Rodriguez, 1994). It has proved to be a relatively stable parameter depending mainly on the ground motion characteristics. In addition, it is moderately to slightly influenced by some structural parameters (period, ductility and hysteretic behaviour) as well. It is practically independent of damping. For approximate determination of γ spectrum, simple formulae were suggested (Fajfar and Vidic, 1994). In a more recent paper (Vidic and Fajfar, 1995) a further simplification was proposed. Sasada et al (1996) derived regression equations for γ spectrum for three groups of records with different peak ground acceleration to velocity ratios.

Equivalent (reduced) ductility factors

An approach including cumulative damage indicators, e.g. hysteretic energy, is not without difficulties in terms of its practical application. A promising technique for handling the problem seems to be the introduction of a new kind of displacement ductility factor which takes into consideration the influence of cyclic load reversals. It is called equivalent (or reduced or weighted) ductility factor. Such an approach represents only a minor adjustment to a concept that is relatively well understood and widely employed in practice. The idea of equivalent ductility factors has already been used in one form or another by several researchers (e.g. Mc Cabe and Hell, 1989, Fajfar, 1992, Negro, 1992, Cosenza et al, 1993). The physical background of equivalent ductility factors is as follows.

Under load reversals well into the inelastic range (beyond a certain critical level) the strength of a structure will deteriorate. The structure will no longer be able to carry the same load at the given deformation level. Load-carrying capacity will continue to decrease during subsequent load cycles until failure takes place. The strength drop-off in each cycle will depend on the amount by which the critical strain has been exceeded. This phenomenon is generally known as low-cycle fatigue. It follows from the above that the deformation capacity of a structure is reduced as a consequence of the dissipation of hysteretic energy caused by cyclic load reversals, and that a substantial reduction in strength can be prevented by limiting the amplitudes of inelastic cyclic deformations. Consequently, a reduced ductility capacity can be defined, which reflects the influence of cyclic response, and which should be used instead of the conventional monotonic ductility capacity in design procedures.

If the Park-Ang damage model is used, the following relation between the reduced ductility (equivalent ductility factor μ_r) and the ultimate monotonic ductility factor μ_u can be obtained (Fajfar, 1992)

$$\frac{\mu_r}{\mu_u} = \frac{DM}{1+\beta\gamma^2\mu_r} \tag{5}$$

where β is a parameter in the Park-Ang model which depends on structural characteristics, mainly on detailing. DM is the permissible damage index. DM larger than 1 represents collapse. From eq. (5) it follows that the larger β and γ, the larger plastic excursions are expected and the lower damage is tolerated, the more is the deformation capacity reduced and, as a consequence, the higher strength is required.

The effect of the cumulative damage can be taken into account in a simple and efficient way if the reduced ductility factor is used instead of the usual ductility factor ($\mu \equiv \mu_r$) in the formulae for R_μ. A suitable damage index can be chosen, allowing the designer to define the acceptable level of structural damage. It is important to realize that, although only few researchers have defined R_μ on the basis of the damage index (e.g. Reinhorn et al, 1992), the reduction factors in the current codes implicitly take into account a certain level of damage, which is supposed to be tolerated after a major earthquake. An average reduction of the ductility capacity due to the cyclic loading is also implicitly considered. For example, according to Part 2 of Eurocode 8, the structure should be capable to sustain a least five full cycles of deformation to the ultimate displacement without excessive deterioration of strength.

Using equivalent ductility factor defined by eq. (5) the explicit use of hysteretic energy is avoided (E_H is implicitly included in γ). The concept has been already implemented in the seismic design metodology developed by Bertero and Bertero (1992).

CONSISTENT INELASTIC SPECTRA

In this section, a procedure for determining consistent inelastic spectra for strength, displacement, hysteretic and input energy is summarized. More details can be find in (Vidic et al, 1994, Fajfar and Vidic, 1994).

For the calculation, ground motion data in terms of elastic (pseudo)acceleration spectrum A_e are needed. In principle, any smooth elastic spectrum can be used. The most convenient is a spectrum of the Newmark-Hall type. For the calculation of energy spectra, the estimated value of $\int \ddot{u}_g(t)\,dt$, or of the duration of the strong ground motion t_D, is needed in addition.

Basic structural parameters, used as input data, are mass, damping, period and tolerated (anticipated) ductility. Equivalent ductility, disscussed in the previous section, can be used in order to include cumulative damage effects and acceptable damage index. Since all spectra are consistent, it is possible to start with known strength and to determine the required ductility. In a displacement based design, the required stiffness can be determined from the known target value for the maximum displacement. Basic relations between spectral values are defined by

$$F_y = \frac{m\,A_e}{R_\mu}, \quad D = \frac{\mu\,A_e}{R_\mu\,\omega^2}, \quad \frac{E_H}{m} = (\gamma\,\omega\,D)^2, \quad \frac{E_I}{m} = \frac{1}{E_H/E_I}\,\frac{E_H}{m} \tag{6}$$

It can be seen that for the determination of the spectra, in addition to A_e, spectra for three non-dimensional parameters (R_μ, γ and E_H/E_I) are needed. The spectra determined according to the summarized procedure, using the proposed approximate values for R_μ, γ and E_H/E_I, simulate quite well the "exact" inelastic spectra determined by nonlinear time-history analysis, as demonstrated in (Vidic et al, 1994, Fajfar and Vidic, 1994).

MDOF SYSTEMS

Inelastic spectra are, in principle, obtained from studies on SDOF systems. They need to be modified for MDOF effects in order to be approximately applicable to real MDOF systems. These modifications are very sensitive to the type of structural system, the fundamental period, and the plastic mechanism (Nassar and Krawinkler, 1991, Seneviratna and Krawinkler, 1994). Much research in this area is still needed.

For evaluation of the seismic behaviour of newly designed or existing MDOF structures, a method based on push-over analysis, equivalent SDOF system and design spectra, e.g. the N2-method (Fajfar and Gašperšič, 1996), can be used.

RECOMMENDATIONS

It is recommended to introduce in the codes the reduction factor R as a product of the ductility dependent factor R_μ and the overstrength factor R_s. A bilinear R_μ spectrum may be assumed. The overstrength factor R_s is to be determined analytically. If not, conservative values for R_s, given in the code for different structural systems, should be used, or even $R_s = 1$. Displacements should be determined by eq.(2). Similar recommendations were made by Uang (1991, 1993) for the codes in the U.S.A. In long term, when more reliable experimental data on strength and deformation degradation due to cyclic loading will be available, it is suggested to use an equivalent ductility factor as a function of the expected structural degradation and of the expected cumulative damage potential of ground motion. Equivalent ductility factor explicitly takes into account the tolerable amount of damage, including the effect of cumulative damage. Hysteretic energy demand, which is responsible for cumulative damage, can be easily predicted by using a spectrum for the parameter γ.

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