First European Conference on Earthquake Engineering and Seismology
(a joint event of the 13th ECEE & 30th General Assembly of the ESC)
Geneva, Switzerland, 3-8 September 2006
Paper Number: 46

DESIGN FORCE RATIO SPECTRUM FOR PERFORMANCE-BASED DESIGN IN CASE OF MULTIPLE EVENTS

Sandip DAS[1], Vinay K. GUPTA[2] and R. Pradeep KUMAR[3]

SUMMARY

The earthquake-resistant design methodology in most existing codes of practice is based on ensuring "no collapse" during the most severe event expected at the given site while most of the input energy is dissipated through inelastic deformations. Evolution of the performance-based design over the last decade has seen a few performance levels added up to this so that the structure remains functional even after a moderately strong event. This methodology however overlooks the possibility that in case of multiple earthquake events expected during the design life of the structure, the structure may get gradually damaged and that it may not be feasible to carry out repairs in the structure after every damaging event. As a result, the structure may collapse earlier than expected and perhaps during an event of moderate intensity. To address such a concern, a new spectrum, called as design force ratio (DFR) spectrum, is proposed in this paper. DFR spectrum gives the ratio by which the design yield force level of a conventionally-designed single-degree-of-freedom structure should be raised such that the damage caused by all earthquake events expected to occur during its lifetime is limited to a specified level. A numerical study is carried out for a hypothetical seismic region by following a simple procedure based on several assumptions, and DFR spectra are obtained for elastic-perfectly plastic oscillators when the return periods of earthquakes follow exponential distribution over the entire range of magnitudes.

1. INTRODUCTION

As per the existing philosophy of earthquake-resistant design, a structure is designed for "no collapse" during the most critical earthquake ground motion expected at the given site. This includes providing strength levels lower than those for the elastic response and then making the structure sufficiently ductile so as to successfully undergo plastic deformations as required by the ground motion. Following the recent developments in performance-based design, some of the codes now ensure that the structure remains functional even after a moderately strong event and thus there are no financial losses due to the disruption of commercial activity during the post-earthquake repairs. This addition of new performance levels does not however discount the possibility that in case of multiple earthquake events expected during the design life of the structure, the structure gets gradually damaged because it may be infeasible to carry out repairs after every event that drives structural response into the inelastic range. The smaller events may even damage the structure and render it unusable well before its design life is over, unless suitable repairs are carried out after every such event causing interruptions in business activity. In an alternate scenario, these events may weaken the structure so much that it can no longer survive the most critical event. The design force level should therefore be so chosen that the damage likely to be experienced by the structure at the end of its design life is limited to a level acceptable to its owner.

[1] *Department of Civil Engineering, Indian Institute of Technology Kanpur, Kanpur, India*
Email : sandip@iitk.ac.in
[2] *Department of Civil Engineering, Indian Institute of Technology Kanpur, Kanpur, India*
Email: vinaykg@iitk.ac.in
[3] *Department of Civil Engineering, Indian Institute of Technology Kanpur, Kanpur, India*
Email: ramancharla@iiit.ac.in

There has been little effort in the direction of evaluating seismic performance of structures during multiple events over a period of time. In a recent study, Amadio and co-workers [Amadio et al., 2003] analyzed the effects of repeated earthquake ground motions on single-degree-of-freedom (SDOF) systems with non-linear behaviour and showed that under repeated earthquake ground motions there was a significant reduction in *q* factor [Ballio, 1990; Vidic et al., 1994] (*q* factor is defined as the ratio between the maximum accelerogram that a structure can withstand without failure and the accelerogram for which the first yielding appears somewhere in the structure). However, they simulated repeated events via repetitions of identical accelerograms, which is clearly incompatible with a realistic seismic environment.

Within the broad framework of performance-based design, this study looks at the factor by which the design yield force level of a conventionally-designed SDOF structure should be raised in order to limit the cumulative damage during its life-time to a specified level. This is done in case of elastic-perfectly plastic oscillators, and a spectrum of this factor, to be called as Design Force Ratio (DFR) spectrum, is proposed when the return periods of earthquake events are exponentially distributed. This spectrum gives the DFR values in case of oscillators of different initial periods. A hypothetical region consisting of four faults with different rates of occurrences is considered for the numerical illustration.

2. DFR SPECTRUM

The design force ratio (DFR) spectrum is defined to give the values of DFR, i.e.

$$\alpha_R = \frac{\overline{Q}_y}{Q_y} \tag{1}$$

for various SDOF oscillators of different initial periods, for a given combination of damping ratio, available ductility and (cumulative) damage level. In Equation (1), Q_y is the yield force level as obtained in the conventional design, and corresponds to the maximum displacement being consistent with the available ductility during the design earthquake event. $\overline{Q}_y$ is the yield force level required for the structure to undergo a specified level of cumulative damage during all earthquake events expected to occur in its lifetime. A value of α_R close to unity implies that the conventional design approach may be adequate at the given site for the structure to survive all earthquake events through its life-time without any post-event repairs, whereas a higher value than unity implies that such an approach may lead to an unsafe design.

For a given oscillator, estimation of Q_y and $\overline{Q}_y$ is an iterative procedure. In any iteration, it is necessary to estimate the largest displacement and damage in the oscillator (of an assumed yield force level) due to an event of given magnitude at a given distance, and to combine damages due to different events while considering event-to-event changes in the oscillator properties due to degradation during the damaging events. Following are the main elements of the procedure used in this study.

2.1 Estimation of Oscillator Response Peaks

We consider the use of linear random vibration theory together with equivalent linearization of the (nonlinear) oscillator for obtaining the largest peak response of the oscillator. Therefore, it may be convenient to characterize the ground acceleration process for an event in terms of its power spectral density function (PSDF). The estimation of PSDF is done here by using the known scaling relationships for Fourier spectrum, strong motion duration, and peak ground acceleration (PGA) in terms of the parameters like earthquake magnitude, epicentral distance and geological site conditions.

Assuming the earthquake ground acceleration to be a stationary process, the PSDF corresponding to the M_k magnitude event occurring at the lth source is evaluated at frequency ω as

$$G_{lk}(\omega) = \frac{Z_{lk}^2(\omega)}{\pi\, T_{lk}} \tag{2}$$

where $Z_{lk}(\omega)$ and T_{lk} respectively are the Fourier spectrum and strong motion duration of ground acceleration for the event of magnitude M_k occurring at epicentral distance R_l. To include the effects of inherent non-stationarity in ground motion, $G_{lk}(\omega)$ is scaled up/down to $\breve{G}_{lk}(\omega)$ so as to correspond to the same expected PGA as that estimated by a suitable scaling relationship. For the scaling of $Z_{lk}(\omega)$, T_{lk} and PGA, the relationships proposed by Trifunac and Lee [Trifunac and Lee, 1985] and Trifunac and Brady [Trifunac and Brady, 1975, 1976] are used.

For an elastic-perfectly plastic SDOF oscillator of initial frequency ω_n, viscous damping ratio ζ and yield displacement x_y, the damping ratio ζ_e and natural frequency ω_e of the equivalent linear oscillator may be obtained as [Caughey, 1960]:

$$\omega_e^2 = \omega_n^2 - \omega_n^2 g(\sigma_y) \tag{3}$$

$$\zeta_e = \frac{\zeta\omega_n}{\omega_e} + \frac{\omega_n^2}{\sqrt{2\pi}\,\omega_e^2\sigma_y}\left[1 - \mathrm{erf}\left(\frac{1}{\sqrt{2}\sigma_y}\right)\right] \tag{4}$$

with

$$\sigma_y = \frac{x_{\mathrm{rms}}}{x_y} \tag{5}$$

$$g(\sigma_y) = \frac{1}{2\pi\sigma_y^4}\int_1^\infty A^3\left(\pi - \Lambda + \frac{1}{2}\sin 2\Lambda\right)\exp\left(-\frac{A^2}{2\sigma_y^2}\right)\mathrm{d}A \tag{6}$$

and erf (.) representing the error function. In Equation (5),

$$x_{\mathrm{rms}} = \left[\int_0^\infty \left|\hat{H}(\omega)\right|^2 \breve{G}_{lk}(\omega)\mathrm{d}\omega\right]^{1/2} \tag{7}$$

is the root-mean-square displacement of the equivalent oscillator where $\hat{H}(\omega)$ is the transient transfer function,

$$\hat{H}(\omega,t) = \frac{1}{(\omega_e^2 - \omega^2) + 2i\zeta_e\omega_e\omega}\left[e^{-i\omega t} - e^{-\zeta_e\omega_e t}\left(\cos\omega_d t + \frac{\zeta_e\omega_e - i\omega}{\omega_d}\sin\omega_d t\right)\right] \tag{8}$$

evaluated at $t = 0.2T_{lk}$ [Gupta and Trifunac, 1998]. Here, ω_d (= $\omega_e\sqrt{(1 - \zeta_e^{\,2})}$) represents the damped natural frequency. It may be noted that the transient transfer function takes care of the effects of non-stationarity in response due to the sudden application of excitation. Also, this function (evaluated at $t = 0.2T_{lk}$) is assumed to be uniformly applicable for the oscillators of all initial periods considered. In Equation (6), we further have

$$\Lambda = \cos^{-1}\left(1 - \frac{2}{A}\right) \tag{9}$$

The linearized properties, ω_e and ζ_e, are obtained through an iterative process requiring an initial estimation of σ_y. Those are then used to obtain the response PSDF as

$$E_{lk}(\omega) = \left|\hat{H}(\omega)\right|^2 \breve{G}_{lk}(\omega) \tag{10}$$

From this, the expected amplitude of the ith order displacement response peak, i.e. $E[x_{(i)}]$, is evaluated as [Gupta and Trifunac, 1988]

$$E\left[x_{(i)}\right] = \left[\int_0^{\infty} E_{lk}(\omega)\mathrm{d}\omega\right]^{1/2} \int_{-\infty}^{\infty} \eta p_{(i)}(\eta)\mathrm{d}\eta \tag{11}$$

where

$$p_{(i)}(\eta) = \frac{N!}{(N-i)!(i-1)!}[P(\eta)]^{i-1}[1-P(\eta)]^{N-i}\,p(\eta) \tag{12}$$

is the probability density function of the ith order peak. Here,

$$p(\eta) = \frac{1}{\sqrt{2\pi}}\left[\varepsilon\, e^{-\eta^2/2\varepsilon^2} + (1-\varepsilon^2)^{1/2}\eta\, e^{-\eta^2/2}\int_{-\infty}^{\eta(1-\varepsilon^2)^{1/2}/\varepsilon} e^{-x^2/2}\mathrm{d}x\right], \tag{13}$$

$$P(\eta) = \int_{\eta}^{\infty} p(u)\mathrm{d}u \tag{14}$$

respectively are the probability density and cumulative probability function of the peaks in the displacement response process, and

$$N = \frac{T_{lk}}{2\pi}\left[\frac{\lambda_4}{\lambda_2}\right]^{1/2} \tag{15}$$

is the expected number of peaks in this process. In Equation (13), ε is the bandwidth parameter defined as

$$\varepsilon = \left[\frac{\lambda_0\lambda_4 - \lambda_2^2}{\lambda_0\lambda_4}\right]^{1/2} \tag{16}$$

where, λ_n is, in general, the nth moment of $E_{lk}(\omega)$ defined as

$$\lambda_n = \int_0^{\infty} \omega^n E_{lk}(\omega)\mathrm{d}\omega, \quad n = 0,1,2,... \tag{17}$$

For the expected amplitude of the ith order peak in the absolute response process, i.e. $E[|x|_{(i)}]$, $p(\eta)$ and N are taken as [Gupta, 2002]

$$p(\eta) = \frac{\sqrt{2}}{\sqrt{\pi}(1+\sqrt{1-\varepsilon^2})}\left[\varepsilon\, e^{-\eta^2/2\varepsilon^2} + (1-\varepsilon^2)^{1/2}\eta\, e^{-\eta^2/2}\int_{-\infty}^{\eta(1-\varepsilon^2)^{1/2}/\varepsilon} e^{-x^2/2}\mathrm{d}x\right], \tag{18}$$

$$N = \frac{T_{lk}}{2\pi}(1+\sqrt{1-\varepsilon^2})\left[\frac{\lambda_4}{\lambda_2}\right]^{1/2}. \tag{19}$$

It may be noted that the properties of the equivalent linear oscillator as obtained via Equations (3)-(9) do not account for the effects of degradation in stiffness or strength during the ground motion. For simplicity, those have been assumed to be negligible. Further, the largest peak amplitude $E[x_{(1)}]$ as computed above is for a chosen value of yield displacement x_y. In conventional earthquake-resistant design, x_y has to be so chosen (for Q_y) that $E[x_{(1)}]$ is equal to μx_y (μ is the available ductility) during the most critical earthquake event likely (at all faults) during the lifetime of the structure. In damage-based design, x_y has to be chosen at a higher level (for $\overline{Q}_y$) so that the oscillator undergoes a specified amount of damage on being subjected to $E[x_{(1)}]$, $E[x_{(2)}]$, …, $E[x_{(N)}]$ response peaks during each of the events expected in the lifetime of the structure. We assume here that the same equivalent oscillator as used for obtaining the largest peak response can be used for obtaining the second largest, third largest, … peak responses also.

2.2 Cumulative Damage during Multiple Events

For the calculation of $\overline{Q}_y$, it is necessary to estimate the damage to which the oscillator of given initial period, damping ratio, and yield displacement will be subjected during the event of magnitude M_k at the lth source. This is estimated here as [Kunnath et al., 1992]

$$D_{lk} = \frac{x_m - x_y}{x_u - x_y} + \beta \frac{EH}{Q_y x_u} \tag{20}$$

where x_m (= $E[x_{(1)}]$) is the maximum displacement that the equivalent linear SDOF system would undergo during the base excitation, x_u (= μx_y) is the ultimate displacement of the system under monotonic loading, β represents the effect of cyclic loading on structural damage, EH represents the total energy dissipation in the structure during the excitation, and Q_y is the lateral force at which first yielding of the structure takes place. Damage D_{lk} is calculated only when x_m exceeds x_y.

Assuming the response to be a narrow-band process and taking $\beta = 0.1$, the damage D_{lk} is obtained as

$$D_{lk} = \frac{E[|x|_{(1)}] - x_y}{x_y(\mu - 1)} + 0.1 \frac{\sum_{i=1}^{N_0} 4\left(E[x_{(i)}] - x_y\right)}{\mu x_y} \tag{21}$$

where, N_0 is the total number of positive zero crossings given by

$$N_0 = \frac{T_{lk}}{2\pi}(1 + \sqrt{1 - \varepsilon^2})\left[\frac{\lambda_2}{\lambda_0}\right]^{1/2} \tag{22}$$

After the damage during each of the events expected in the design life of the structure is estimated, the cumulative damage in that period is estimated by adding individual damages caused by those events. This study considers the most conservative estimate of the cumulative damage, and therefore, all anticipated events are assumed to take place in the descending order of the damage they would inflict on the just-built structure. As the events that cause no damage to the undamaged structure may still cause some damage to the structure with degraded strength and stiffness, those are assumed to take place in the descending order with respect to the maximum displacement of the just-built structure.

It is assumed that the yield displacement $x_{y,i}$ increases after the ith damaging event to $x_{y,i+1}$, such that

$$x_{y,i+1} = \left(\frac{k_1 + k_i}{k_1 + k_{i+1}}\right) x_{y,i} \tag{23}$$

Here, k_1 denotes the initial stiffness of the system, and k_i and k_{i+1} respectively denote the stiffness of the system before and after the ith event. Equation (23) is based on the assumption that the maximum increase in the yield displacement is equal to $x_{y,1}$ (= x_y) in the case of complete degradation of the stiffness. Further, considering the stiffness degradation to be an increasing function of the maximum displacement beyond the yield displacement during the ith damaging event, the stiffness k_{i+1} is assumed as

$$k_{i+1} = k_i \left[1 - \frac{E[|x|_{(1),i}] - x_{y,i}}{x_{u,i} - x_{y,i}}\right]^{\gamma} \tag{24}$$

Here, $|x|_{(1),i}$, $x_{y,i}$, and $x_{u,i}$ respectively represent the values of $|x|_{(1)}$, x_y, and x_u during the ith damaging event. Further, the parameter, γ, is taken arbitrarily as 0.1 considering the degradation in the moment capacity of the reinforced concrete members as modelled by Reinhorn and co-workers [Reinhorn et al., 1992]. The strength degradation is assumed to be governed by the stiffness degradation as in Equation (24) and by the increase in yield displacement as in Equation (23). It is also assumed that the structure does not undergo any repairs after any damaging event.

3. EXPECTED MAGNITUDES OF EARTHQUAKE EVENTS

For the calculation of the cumulative damage due to various seismic events, it is necessary to know the number of events of different magnitudes that are likely to occur at each of the faults in a given time-window. Let the seismicity of each contributing fault be known apriori in terms of the rates of occurrences of earthquakes of different magnitudes. Further, let the average rate of occurrence per year, N_l, of the earthquakes of magnitude M and higher at the lth source be described as [Gutenberg and Richter, 1942]

$$\log N_l = a_l - b_l M \tag{25}$$

where a_l and b_l are constants estimated from the catalog of past earthquakes or from the known slip rate at the lth source. It is assumed that all the events of magnitude M and higher at a source follow the exponential distribution of return period. Therefore, the hazard rate becomes constant and equal to N_l. In this study, events below magnitude 5 are neglected, and it is assumed that no event with magnitude $M > 8$ will occur.

After calculating the expected number of earthquakes for a fault, the maximum magnitude during a given period of time may be estimated for a confidence level by the extreme event analysis. As there is no method available to estimate the magnitudes of higher order events, the expected value of the magnitude of the ith largest event is taken as

$$E[M_i] = \int_{M_{\min}}^{M_{\max}} m\, p_i(m)\,\mathrm{d}m \tag{26}$$

assuming statistical independence between the events likely to occur at a fault for a given period of time. Here

$$p_i(m) = \frac{\tilde{N}!}{(\tilde{N}-i)!(i-1)!}[F(m)]^{i-1}[1-F(m)]^{\tilde{N}-1}\,p(m) \tag{27}$$

is the probability density function of the ith largest event according to the order statistics approach. In Equation (27), $\tilde{N}$ is the expected number of events in Y years in the range $M_{\min}$ (= 5) to $M_{\max}$ (= 8). Further,

$$F(m) = \frac{10^{-b(m-M_{\min})} - 10^{-b(M_{\max}-M_{\min})}}{1-10^{-b(M_{\max}-M_{\min})}} \tag{28}$$

$$p(m) = b\ln 10 \frac{10^{-b(m-M_{\min})}}{1-10^{-b(M_{\max}-M_{\min})}} \tag{29}$$

respectively denote the probability distribution and density functions of $\tilde{N}$ events.

4. NUMERICAL STUDY

For the purpose of obtaining DFR spectra (i.e., plots of α_R with the initial period of the SDOF oscillator), a hypothetical seismic environment with faults of known parameters, a and b, has been considered in this study. This seismic environment is similar to that considered by Todorovska [Todorovska, 1994] and consists of four faults: two faults located at a distance of 30 km each from the site, and the other two located at 40 and 50 km each. The values of a_l for these faults are taken as 3.28, 4.03, 3.77 and 3.09, respectively, while b_l has been assumed to be uniformly equal to 0.86 for all the four faults. The focal depths of the sources are assumed to be uniformly equal to 5 km, and the area under consideration is assumed to have alluvium geologic site conditions.

Twenty elastic-perfectly plastic oscillators with initial periods of 0.1, 0.15, 0.2, 0.25, 0.3, 0.4, 0.5, 0.6, 0.75, 0.9, 1.0, 1.1, 1.25, 1.5, 1.75, 2.0, 2.5 s and uniform damping ratio of 5% are considered. The DFR spectra are constructed for a design life $Y = 50$ years. The value of Q_y in each case is determined by considering the lowest yield strength such that the maximum displacement of the equivalent linear oscillator does not exceed μx_y during the largest magnitude event expected (for $i = 1$ in Equation (26)) at each of the four faults during the lifetime of the structure.

Figure 1 shows the DFR spectra for $\mu = 2.0$, 2.5 and 3.0 in case of cumulative damage level $D = 0.8$ (see the curves without cross symbols). It is assumed that $D = 0.8$ is representative of critical damage for collapse as envisaged in the traditional seismic design for the most severe event. Figure 1 also shows the strength reduction factor R_μ values for the three ductility levels (see the curves with cross symbols). R_μ here refers to the ratio of the required yield strength for unit ductility demand to that for μ ductility demand in the example seismic environment. If the most critical events are identical for the elastic and inelastic responses, R_μ would become same as the conventional strength reduction factor. It may be observed that α_R increases with increasing ductility for a given initial period of oscillator but for a given ductility ratio, this remains below R_μ. As will be shown in Figure 2, α_R takes higher values for lower D values. At $D = 0$, the maximum displacement may not exceed the yield displacement under the most critical excitation, and then α_R would become same as R_μ. It is obvious from Figure 1 that (conventionally-designed) more ductile structural systems may be more prone to failing to survive through the design life of the structure than the less ductile systems. This is because the conventional design approach envisages greater reductions in yield strength levels for more ductile systems, leading to higher damage levels and thus leaving little margins for surviving the other events.

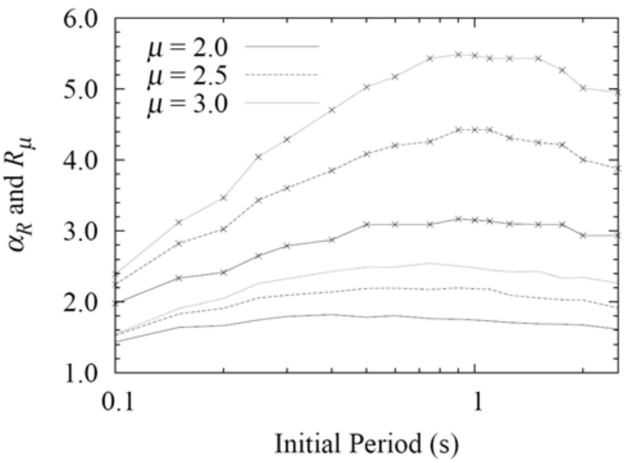

Figure 1: DFR spectra (without cross symbols) and R_μ spectra (with cross symbols) for different values of ductility ratio μ in case of $D = 0.8$

Figure 2 shows the DFR spectra for (cumulative) damage levels of $D = 0.4$, 0.6 and 0.8 in case of $\mu = 3$. The additional damage levels of 0.4 and 0.6 here represent the cases of moderate and significant damages in the structure, respectively. Though these damage levels are not consistent with the desired state of structure at the end of its design life, Figure 2 clearly shows the effect of damage level on the DFR spectrum. As expected, α_R increases with decrease in the damage level for a given oscillator period. It may be observed from Figure 2 that the yield levels obtained from the conventional design are not good enough for the multiple-event design, particularly when we aim for cumulative damage much less than that for the collapse. Those may however be acceptable when the seismic environment becomes much milder than that considered in the example case here. This is illustrated in Figure 3 where DFR spectra for $\mu = 3$ and $D = 0.8$ are compared for three seismicity levels: (i) 'Seism1' for the seismicity level of the example case, (ii) 'Seism2' for the seismicity level corresponding to 50% of the earthquake occurrences in 'Seism1', and (iii) 'Seism3' for 5% of the earthquake occurrences in 'Seism1'. The cases of 'Seism2' and 'Seism3' have been obtained by taking the values of a_l as (2.98, 3.73, 3.47, 2.79) and (1.98, 2.73, 2.47, 1.79), respectively. It may be mentioned that the case of 'Seism3' effectively corresponds to single-event design. However, the α_R levels here are not close to unity because α_R includes the effects of shift from ductility-based to damage-based design. The effects of this shift can be made negligible by choosing a value of D suitably higher than 0.8.

Figure 4 shows the variation of DFR spectrum with design life, Y. As expected, the value of α_R increases with an increase in the design life. In other words, in case of longer design life the structure will be exposed to seismic activity for a longer time duration and will thus experience a greater number of damaging events.

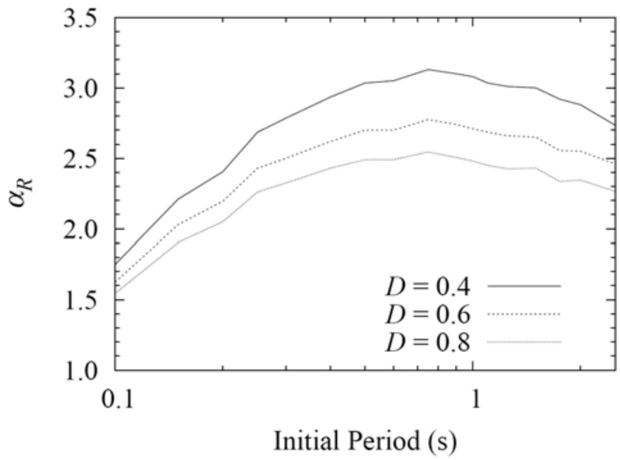

Figure 2: DFR spectra for different values of cumulative damage *D* in case of μ = 3

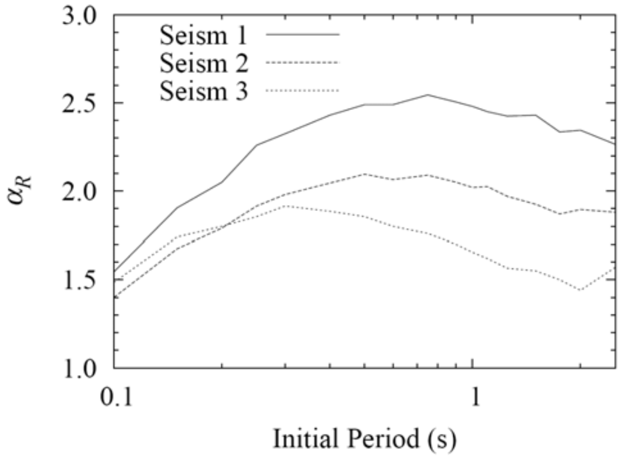

Figure 3: DFR spectra for different levels of seismicity in case of μ = 3 and *D* = 0.8

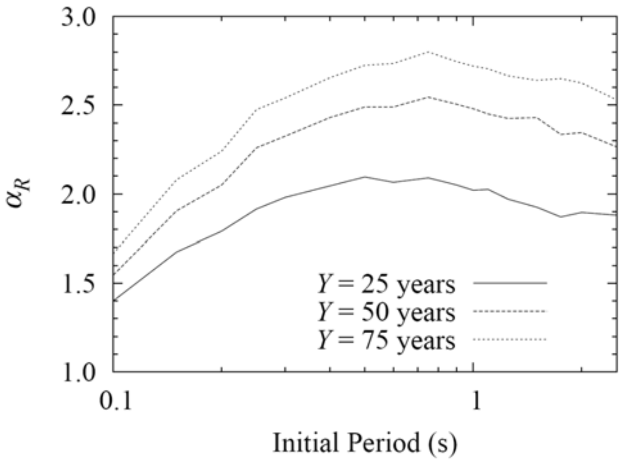

Figure 4: DFR spectra for different values of design life *Y* in case of μ = 3 and *D* = 0.8

5. CONCLUSIONS

In this study, the concept of DFR spectrum has been proposed for raising the yield strength level of a conventionally-designed SDOF structural system such that it survives all earthquake events during its design life without having to undergo any repairs after the damaging events. Besides accounting for the effects of multiple events, DFR also accounts for the effects of disparities between the ductility-based and damage-based designs. The calculations for DFR spectrum are based on several assumptions, some well-accepted and some made afresh in this paper. Still, the results obtained are broadly consistent with the known trends. The concept of DFR spectrum may thus mark a useful step in the direction of achieving a more comprehensive performance-based design with no post-earthquake repairs.

6. REFERENCES

Amadio, C., Fragiacomo, M. and Rajgelj, S. (2003), The Effects of Repeated Earthquake Ground Motions on the Non-Linear Response of SDOF Systems, *Earthquake Engineering and Structural Dynamics*, 32, 291-308.

Ballio, G. (1990), European Approach to Design of Steel Structures, *Proceedings of Hong Kong Fourth World Congress*, Hong Kong, 935-946.

Caughey, T.K. (1960), Random Excitation of a System with Bilinear Hysteresis, *Journal of Applied Mechanics, Transactions of ASME*, 27, 649-652.

Gupta, I.D., Trifunac, M.D. (1988), Order Statistics of Peaks in Earthquake Response, *Journal of Engineering Mechanics, ASCE*, 114, No. 10, 1605-1627.

Gupta, I.D., Trifunac, M.D. (1998), Defining Equivalent Stationary PSDF to Account for Nonstationarity of Earthquake Ground Motion, *Soil Dynamics and Earthquake Engineering*, 17, 89-99.

Gupta, V.K. (2002), Developments in Response Spectrum-Based Stochastic Response of Structural Systems, *ISET Journal of Earthquake Technology*, 39, No. 4, 347-365.

Gutenberg, B., Richter, C.F. (1942), Earthquake Magnitude, Intensity, Energy and Acceleration, *Bulletin of the Seismological Society of America*, 32, 163-191.

Kunnath, S.K., Reinhorn, A.M. and Lobo, R.F. (1992), IDARC Version 3.0: A Program for the Inelastic Damage Analysis of RC Structures, *Report NCEER-92--2002, National Center for Earthquake Engineering Research, State University of New York at Buffalo*, New York.

Reinhorn, A.M., Kunnath, S.K. and Mander, J.B. (1992), Seismic Design of Structures for Damage Control, In *Nonlinear Seismic Analysis and Design of Reinforced Concrete Buildings*, Fajfar, P., Krawinkler, H. (eds), *Elsevier Applied Science*, London.

Todorovska, M.I. (1994), Comparison of Response Spectrum Amplitudes from Earthquakes with a Lognormally and Exponentially Distributed Return Period, Soil Dynamics and Earthquake Engineering, 13, 97-116.

Trifunac, M.D., Brady, A.G. (1975), A Study on the Duration of Strong Earthquake Ground Motion, *Bulletin of the Seismological Society of America*, 65, 581-626.

Trifunac, M.D., Brady, A.G. (1976), Correlation of Peak Acceleration, Velocity and Displacement with Earthquake Magnitude, Distance and Site Conditions, *Earthquake Engineering and Structural Dynamics*, 4, 455-471.

Trifunac, M.D., Lee, V.W. (1985), Preliminary Empirical Model for Scaling Fourier Amplitude Spectra of Strong Ground Acceleration in Terms of Earthquake Magnitude, Source to Station Distance, Site Intensity and Recording Site Conditions, *Report No. CE 85-03, Department of Civil Engineering, University of Southern California*, Los Angeles, California.

Vidic, T., Fajfar, P. and Fischinger, M. (1994), Consistent Inelastic Design Spectra: Strength and Displacement, *Earthquake Engineering and Structural Dynamics*, 23, 507-521.