Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Variable safety factors for seismic design of RC

M. N. Fardis & S. N. Economou
Department of Civil Engineering, University of Patras, Greece

ABSTRACT: A procedure is proposed and applied for the determination of variable safety factors for the combination of seismic and gravity loads, to achieve uniform probability of failure under bending with axial load. Safety factors depend on the type of member, on the prescribed probability of failure and on the ratio of the load effect due to gravity loads to that due to the nominal value of the seismic action, both obtained from elastic analysis. They are computed through a Level II Reliability procedure, using a Limit State inequality between the member rotation ductility supply under monotonic loading and the peak rotation ductility and cyclic energy dissipation demands. Uncertainties considered are: On the resistance side, that expressing the uncertainty of failure under cyclic loading, and on the action side that in the maximum peak ductility and energy dissipation demands in the structure's lifetime, as obtained by Monte-Carlo simulations of the maximum spectrum in this lifetime and the corresponding ground motion, in conjuction with nonlinear dynamic analyses.

1. INTRODUCTION

According to modern seismic design codes, dimensioning of elements on the basis of the Ultimate Limit State to longitudinal load effects (bending moment and axial force) is performed using the combination of the quasi-permanent or arbitrary-point-in-time gravity loads (dead and live) and the characteristic value of the seismic action, with partial safety factors equal to 1.0 in the European Codes or to a constant value greater than 1.0 in the UBC. Design seismic load-effects (M, N) are obtained by dividing the elastic ones by the "behaviour" or "response modification" factor, which accounts semi-empirically for the inelastic energy dissipation, for the effects of non-structural elements, etc. It is commonly recognised that this design format provides a nonuniform and unknown probability of failure for the elements and structures so designed. In this paper an alternative design procedure is developed, in which partial safety factors on the resistance, on the quasi-permanent values of gravity loads and on the seismic action are not constant but depend on a) the ratio of elastic load-effects (bending moments) due to the quasi-permanent value of gravity loads to that due to the nominal value of the seismic action; b) on the acceptable probability of failure for the element and structure in question; and c) on the type of the member (beam, column, etc.). This procedure is along the lines of the work which led to probability based load factors for American National Standard A58 (Ellingwood et al, 1980). Although the present scope is narrower than that of this previous work, as it is limited to the design of reinforced concrete for the combination of earthquake and gravity loads, the present approach is much more general, as a) it aims at a uniform probability of failure, controlled by non-constant partial safety factorss, and b) it is based on a realistic and well-callibrated criterion for earthquake-induced member ultimate failure, rather than on the arbitrary adoption of a constant "behaviour" or "response modification" factor, as a means for translating elastically calculated earthquake-induced load-effects to member ultimate failure.

2. LIMIT STATE EQUATION FOR FAILURE UNDER EARTHQUAKE - INDUCED DEFORMATIONS

The aim of Ultimate Limit State (ULS) design of a member or a structure is to achieve a target value of its conventionally defined Reliability, or equivalently, of its complement, i.e., of the Probability of Failure within its conventional lifetime, P_f. Within the framework of Level I or II Reliability Theory, the magnitude of P_f is controlled through the Reliability Index β, which is the minimum distance of the "surface" $g(x_i) = 0$ which describes analytically the ULS of interest in terms of the n relevant normalized random variables x_i (i = 1, n) from the origin $x_i = 0$, in the space of the x_i.

The normalization of the x_i is into a set of uncorrelated random variables, each with zero mean and unit variance.

The minimum distance point of $g(x_i) = 0$ is termed "design point", x^*_i (i = 1, n), and is the most likely failure point. Its coordinates equal $x^*_i = -a_i\beta$ in which a_i is the projection of the unit normal of $g(x_i) = 0$ at x^*_i onto the x_i axis, and can be translated to the values $X^*_i = X_i - a_i \beta \sigma_i$ of the original unnormalized (but mutually uncorrelated) variables X_i, which have mean X_i and variance σ^2_i (i = 1, n). X^*_i is conventionally expressed as the product of the corresponding partial safety factor γ_i and the conventionally defined nominal value X_{ni} of X_i: $X^*_i = \gamma_i X_{ni}$. So, the target value of P_f is achieved through the selection of the partial safety factors γ_i so that all the $X^*_i = \gamma_i X_{ni}$ result from the value of β which

corresponds to P_f (For normal probability distributions of the x_i and X_i, the values of β corresponding to P_f = 10^{-1}, 10^{-2} or 10^{-3}, etc., are 1.28, 2.32 and 3.09, etc.). Depending on how the nominal values X_{ni} are conventionally defined, γ_i may assume values less than 1.0, if X_i is a "resistance" variable, such as the strength of concrete or steel, etc. (cf. the "capacity reduction" factors of ACI-318 and the inverse of the material safety factors of the Eurocodes), or greater than 1.0, if X_i is a load or action variable. In the special case of seismic design, in which load-effects due to the nominal or characteristic seismic action are conventionally determined through a linear elastic analysis although the response to this action is expected to be well in the inelastic range, the γ_i on the nominal value of the seismic action roughly corresponds to the inverse of the "behaviour factor" q of the Eurocodes or of the "response modification factor" of ATC-3, and assumes values well below 1.0.

In structures consisting of one-dimensional or linear elements, such as beams, columns and shear walls, the ULS is conveniently defined and verified at the cross-sectional level, with the X_i being action-induced load effects (M, V, N) and the corresponding resistances of the cross-section. Moreover, as reinforced concrete members are designed in shear for internal forces determined by capacity design rules and not from the load-effects due to the nominal seismic action, only the ULS of such members to longitudinal load-effects (M, N) is verified according to the Level I or II partial safety factor approach outlined above. This verification usually assumes the convenient linear form:

$$\gamma_g M_{SGn} + \gamma_E M_{SEn} \geq \gamma_R M_{Rn} \tag{1}$$

in which M_{SGn} and M_{SEn} are the bending moments at the cross-section of interest due to the nominal value of the gravity loads (G) and seismic action (E) and γ_g, γ_E the corresponding load factors, M_{Rn} is the corresponding nominal capacity (resistance) and γ_R the associated (capacity reduction or resistance) safety factor.

In this study, the convenient traditional verification form of Eq. (1) is retained, and the partial safety factors γ_R on the nominal flexural capacity M_{Rn} (taken here equal to M_{Rd}, i.e. to the design flexural capacity of the European codes, in which the material safety factors for steel and concrete, γ_s = 1.15 and γ_c = 1.5, are incorporated in the associated design strengths f_{yd} and f_{cd}), and γ_g and γ_E on the elastically determined load-effects M_{SGn} and M_{SEn} are determined as functions of the ratio M_{SEn}/M_{SGn} and of the reliability index β. To this end, an expression describing failure of reinforced concrete members under imposed cyclic deformations must be adopted for the ULS equation, $g(X_i) = 0$, and expressed in terms of the design flexural capacity, M_{Rd}, and of the load-effects of the gravity loads, M_{SG}, and of the seismic action, M_{SE}, both as determined by a linear-elastic analysis.

Several criteria have been proposed to define failure of reinforced concrete members under cyclic deformations, such as those induced by seismic actions. A commonly used failure criterion is the exceedance of the available displacement, rotation or maximum curvature of the member, as the latter is determined by the help of empirical or semi-empirical expressions in terms of the geometric and mechanical characteristics of the member, and its reinforcement. Cumulative damage measures, such as the hysteretic energy dissipation, have also been proposed, along with empirical expressions for their supply values (e.g., Darwin and Nmai, 1986). Park and Ang (1985) have proposed a linear combination of the peak displacement ductility demand and of the hysteretic dissipated energy, both defined over the shear span of the member, as a comprehensive failure criterion under cyclic deformations:

$$\frac{\delta_{max}}{\delta_y} + \frac{\beta_{PA}}{P_y \delta_y} \int P d\delta = \varepsilon_{PA}\, \mu_{u,PA} \tag{2}$$

in which δ is displacement of the point of inflection with respect to the tangent at the end section of the member, P the shear force, considered constant over the shear span, and δ_y, P_y the corresponding values at flexural yield of the end-section.

Park and Ang (1985) have developed a semi-empirical procedure for the calculation of δ_y, taking into account contributions due to flexural and shear deformations and bond-slip effects, and fitted an empirical expression for the calculation of the corresponding ductility supply, $\mu_u = \delta_u/\delta_y$ at monotonic failure, to a set of 142 monotonic tests. The value of μ_u obtained from this expression is denoted by $\mu_{u,PA}$ in Eq. (2). The same authors developed an empirical expression for the proportionality coefficient β, on the basis of the results of 261 cyclic tests to failure. The so-computed value of β is denoted in Eq. (2) by β_{PA}. With the values of β_{PA} and $\mu_{u,PA}$ obtained through the Park and Ang (1985) procedure, Eq. (2) fits the data in the 403 monotonic or cyclic tests to failure of the Park and Ang data bank, albeit with a lognormally distributed scatter term, $\varepsilon_{PA} \sim LN\,(1.008, 0.535^2)$. This latter databank includes members in uniaxial bending with or without axial load, with a wide range of shear span ratios.

The predictions of Eq. (2), as well as those of several other monotonic (Tassios, 1988a, 1988b, Keintzel, 1986, Lybas and Sozen, 1977) or cyclic (Darwin and Nmai, 1986) failure criteria, have been compared, within the framework of the present study, to a set of 28 cyclic tests to failure, essentially outside the data bank used by Park and Ang (1985). The failure criterion of Eq. (2) was found as the only one coming close to the failure data of this smaller data bank, and indeed with an average fit and a measure of the scatter not far from those given by the Park and Ang scatter term, ε_{PA}. So, it was decided to base on Eq. (2) the development of equation $g(X_i) = 0$, which describes the ULS of reinforced concrete members in uniaxial bending with or without axial load. To this end, Eq. (2) is algebraically transformed into the following, which is of the form of Eq. (1):

$$\varepsilon_R M_{Rd} = \pm M_G + \varepsilon_E M_{SEn} \tag{3}$$

in which:

$$\varepsilon_R = \varepsilon_{PA} \frac{\mu_{u,PA} M_y}{M_{Rd}} \tag{4}$$

$$\varepsilon_E = \frac{\frac{6EI}{l}\left(\delta_{max} + \beta_{PA} \int \frac{M}{M_y} d\delta\right) \mp M_G}{M_{SEn}} \tag{5}$$

In Eqs. (4) and (5), the yield moment, M_y and the design value of the flexural capacity, M_{Rd}, refer to the end-section, and are computed conventionally for the value of axial force due to gravity loads alone, θ is the end rotation with respect to the chord connecting the two ends, assuming antisymmetric bending (point of inflection at l/2), θ_{max} is the peak value over the inelastic seismic response, and M_G, M_{SEn} are the bending moments at the end in question as computed from a conventional linear elastic (usually static) analysis for the nominal values of the arbitrary-point-in-time or quasi-permanent gravity loads, G, and of the seismic action, E, respectively.

Denoting the term in the LHS of Eq. (2) by X_1 and the 1st and 2nd term in its RHS by X_2 and X_3, the equation describing the ULS of flexure-controlled failure under imposed cyclic deformations is of the form:

$$g(X_i) \equiv -X_1 \pm X_2 + X_3 = 0 \qquad (6)$$

For the determination of the partial safety factors γ_i (i = 1, 2, 3) to be applied on the nominal values of X_i, X_{ni}, these latter nominal values should be specified, and the probability distributions of X_i must be known. These latter are determined as follows:

1) Considering M_{Rd} as the nominal value of the "resistance" random variable, X_1, the distribution of the latter is determined from that of ε_R, which in turn reflects: a) the scatter about the Park and Ang (1985) failure criteria for given mechanical and geometric characteristics of the member, expressed by the aforementioned probabilty distribution of $\varepsilon_{PA} \sim LN(1.008, 0.535^2)$; and b) the variability of $\mu_{u,PA}\ M_y/M_{Rd}$ within the population of beams or columns, etc., designed and detailed according to a modern seismic code. This variability was established by i) designing and detailing several hundreds of beams and columns according to the (draft) Eurocodes 8 (Duct. level M) and 2, from various buildings designed for different levels of seismic action intensity, ii) simulating in a Monte-Carlo fashion the mechanical properties of steel and concrete in these members from the corresponding probability distributions, iii) computing for each member the values of $\mu_{u,PA}$, M_y and M_{Rd}, the latter two using conventional mechanical models, and iv) fitting a probability distribution to the histogram of $\mu_{u,PA}\ M_y/M_{Rd}$ over all these members. This distribution was found to be approximately lognormal, but with different moments for columns, and for beams in positive or negative bending (tension at the bottom of the web, or at the top flange, respectively). At the end, using the properties of the lognormal distribution, the following distributions of ε_R were determined:

For columns: $\varepsilon_R \sim LN(13.7, 8.7^2)$ (7a)

For (T) beams in positive bending:

$\varepsilon_R \sim LN(57.3, 39.9^2)$ (7b)

For beams in negative bending:

$\varepsilon_R \sim LN(20.4, 15.1^2)$ (7c)

The three distributions in Eqs. (7) were found to have similar coefficients of variation, so, far simplicity, an average value equal to 0.69 was adopted for all three.

Physically ε_R is the product of the available rotation (or displacement) ductility factor, in monotonic loading, times the ratio of actual yield moment to design flexural resistance. This ratio incorporates the effect of the material safety factors for steel, γ_s = 1.15, and concrete, γ_c = 1.5, as well as the difference between actual and characteristic values of the strengths. It can be considered as the aggregate safety factor built in the design flexural capacity of the member, as an estimate of its yield moment. Approximately half of the above variances of ε_R reflects the uncertainty of the Park and Ang (1985) ductility factor as an estimator of member failure in monotonic loading. The differences in the mean value of ε_R reflect inherent differences in the ductilities of the three cases above, i.e., the lower ductility of columns, due to the axial load and the high ductility of T-beams in positive bending.

2) For the "gravity loads" variable X_2, we need the probability distribution of the ratio of M_G to its nominal value M_{SGn}. This distribution can be taken as normal, with mean equal to 1.05 (due to the tendency of the designer to underestimate permanent and quasi-permanent gravity loads), and variance 0.15^2, which mainly reflects the model uncertainty in the determination of load-effects for given nominal values of the gravity load.

3) Random variable X_3 reflects the uncertainty in the seismic action and in the structural response to it. The procedure for the establishment of its probability distribution involves several computational steps on the same buildings used to compute the probability distribution of ε_R: a) The (positive and negative) bending moments computed at the ends of the members in the course of the linear-elastic analysis for the nominal seismic action are identified as M_{SEn}. If this latter action incorporates the "behaviour" or "response modification" factor q, the analysis results are multiplied by q to obtain the values of M_{SEn}. b) An ensemble of pairs of ground motions in the two horizontal directions is artificially generated, simulating the extreme seismic event in the building's conventional lifetime at the corresponding site (see next chapter for details). c) A series of nonlinear dynamic analyses of each of the buildings is performed in 3D, using as input to each analysis a pair of horizontal ground motions from the above ensemble. Each analysis provides two values of the sum in parenthesis in Eq. (5) for each end of every member of the structure, one for positive and another for negative bending. These values, divided by the corresponding value of M_{SEn} at the end in question, produce corresponding values of ε_E, through Eq. (5). d) A probability distribution is fitted to the histogram of the several thousands of so-computed ε_E values, separately for columns and for beams in positive or in negative bending. These probability distributions reflect the uncertainty i) in the extreme seismic event during a structure's conventional life-time, and ii) in the nonlinear dynamic response to this event, for given nominal seismic action.

The nonlinear dynamic response analyses are performed using a version of the program ANSR-I, developed at U.C. Berkeley by Mondkar and Powell (1975), with a beam and beam-column element developed at the Univ. of Patras. The element is of the point-hinge (lumped inelasticity) type in flexure, with antisymmetric bending and the modified Takeda model for the end moment - plastic hinge rotation relationships (bilinear primary curve and overall 9 rules for the hysteresis, as proposed by Litton (1975)), separately and independently for each transverse direction of bending. It includes also inelasticity and degradation of stiffness and strength in shear, according to a shear model proposed by Fardis (1991). For consistency with the

damage and failure models used in Eqs. (2) and (5), the deformations at the yield and ultimate strength points of the multilinear primary curves in flexure and in shear are computed on the basis of the corresponding Park and Ang (1985) models, which account separately for deformations due to shear and longitudinal action effects (flexure and bond-slip of the longitudinal bars).

The probability distributions fitted to the histograms of ε_E are Extreme Value of Type II, with the following parameters, which were found independent of the sign of the erathquake-induced inelastic deformations relative to that of the gravity load-effects (the +/- sign in Eq. (5)):

For columns:

u = 0.33, k = 2.22 (i.e. mean: 0.53, c.o.v.: 1.63).

For beams, in negative or in positive bending:

u = 0.33, k = 2.7 (i.e. mean: 0.47, c.o.v.: 0.84).

Physically ε_E is the ratio of the extreme (over the structure's lifetime) earthquake-induced member deformations (rotations) to the elastically computed corresponding deformations due to the nominal earthquake action, which does not include a "response modification" or "behaviour" factor. Its mean value is about equal to the ratio of the expected value of the maximum (over the structure's lifetime) mean (over a wide range of periods) elastic response spectral acceleration, to the spectral acceleration used to compute the nominal seismic action (design ground acceleration, times the spectral amlification factors without "response modification" or "behaviour" factor). The type of its probability distribution and its variance are dictated mainly by those of the extreme ground and elastic response accelerations in the structure's lifetime.

3. MONTE-CARLO SIMULATION OF MAXIMUM EARTHQUAKE EXCITATION IN A STRUCTURE'S LIFETIME

The bidirectional ground motions used as input to the series of nonlinear dynamic response analyses are meant to be random realizations of the maximum earthquake excitation in the structure's lifetime, conventionally considered to be 50 yrs. They are generated as pairs of artificial motions for the two horizontal directions, matching corresponding elastic response spectra which are drawn from an ensemble of 20 pairs of irregular spectra for each site of interest. The ensembles of pairs are generated in a Monte-Carlo fashion, so that the correlation between the two horizontal components is preserved, and the sample mean, variance and autocorrelation functions match the corresponding functions of the maximum spectrum in a structure's lifetime. The Monte-Carlo simulation is based on the probability distributions of the maximum elastic response spectral accelerations for a set of 11 discrete values of natural period (including the value for T = 0 sec., i.e., for the peak ground acceleration), and ranging up to 1.5 sec.), and on an empirically derived auto-correlation function for the spectral ordinates at different periods, normalized to the peak ground acceleration. As described in detail by Economou and Fardis (1992), the peak ground acceleration is simulated from its own probability distribution (an Extreme Value of Type II, established as described in the sequel), and independently a set of 10 uncorrelated random variables is simulated (each one for the 10 discrete T-values chosen for the description of the spectrum), consistently with the probability distribution of the corresponding normalized spectral value, and then linearly transformed into a set of 10 correlated variables, with the desired mean vector and autocovariance matrix, as the latter is obtained from the target variance and autocorrelation functions.

The marginal distributions of the maximum over T_L years peak ground acceleration and response spectral values at the 10 discrete values of the period (from which the marginal distributions of normalized spectral values are obtained by appropriate algebraic transformations), are well approximated by Extreme Value distributions of Type II. This distribution results from the usual Poisson assumption of occurrences of seismic events, in conjunction with a linear fit in log-log of the Seismic Hazard results for the mean annual rate of exceedance of spectral acceleration $S_a(T)$, $\lambda_{SA}(T)$, as a function of $S_a(T)$:

$$\ln\lambda_{SA}(T) \approx \ln\lambda_1 - k \ln S_a(T) \tag{7}$$

resulting in:

$$F_{SA,max}(S_a(T)) \equiv P[S_{a,max}(T) \le S_a] = = e^{-\lambda_{SA}(T)T_L} \approx e^{-\lambda_1 T_L S_a^{-k}} \tag{8}$$

Parameters λ_1 and k of this extreme Type II distribution are obtained by requiring that its mean value and coefficient of variation, which are equal to $(\lambda_1 T_L)^{1/k}\,\Gamma(1-1/k)$ and $(\Gamma(1-2/k)/\Gamma^2(1-1/k)-1)^{1/2}$ respectively, equal to the numerically obtained mean value and coefficient of variation of the exact distribution of $S_a(T)$, given by the second equality in Eq. (8).

The curves of mean exceedance rate $\lambda_{SA}(T)$ vs. Spectral acceleration $S_a(T)$ at period T can be obtained by a Seismic Hazard Analysis on response spectral values. For this particular study the Seismic Hazard Analysis was based on two alternative attenuation laws on ground and spectral accelerations: The ones by Trifunac (1976), and Trifunac and Anderson (1977), and the earlier one by McGuire (1974). A weighted average of the results obtained from the two alternative attenuations was used, with a weight of 2. given to the Trifunac-based results vs. a weight of 1. for the McGuire ones. As described in detail by Economou and Fardis (1990) the Seismic Hazard Analysis was applied to 4 cities in Greece, and accounted for statistical uncertainty in the geometry (location of the sides) of the quadrilateral seismic sources, and in the slope and the upper-bound Magnitude of the Magnitude-recurrence relation.

4. RESULTS AND CONCLUSIONS

The procedure described above was applied to three 6-storey buildings, one in each of the cities of Athens, Salonica and Patras, designed for the corresponding design spectra, and each nonlinearly analysed for an ensemble of 20 pairs of horizontal ground motion components, each ensemble being representative of the extreme earthquake excitation at the corresponding site. The probability distributions of ε_E quoted above are obtained from these nonlinear dynamic analyses of the three buildings. Then, the Level II algorithm proposed by

Ellingwood et al (1980) was applied to determine to partial safety factors γ_g, γ_E and γ_R in Eq. (1), required to obtain a uniform notional probability of failure $P_f = 10^{-1}$ in 50 yrs. for the member (Reliability index $\beta = 1.282$). Although this level of Reliability sounds low, it is higher than that calculated by Ellingwood et al (1980) for reinforced concrete and hot-rolled steel beams and columns in Los Angeles, designed for the combination of earthquake and gravity loads prior to the new American National Standard A58: "Building Code Requirements for Minimum Design Loads in Buildings and Other Structures" (1984). It is also not much lower than that resulting from the constant load factors and load combinations of the above-mentioned new American National Standard, which aim at $\beta = 1.75$ i.e. at a notional probability of failure $4x10^{-2}$ on the average (Ellingwood et al, 1980). Numerical problems arising from the high coefficients of variation of the resistance and the action terms, ε_R and ε_E, have not allowed as yet the application of the algorithm to compute the partial safety factors required to achieve significantly higher Reliablibity levels, (e.g. $P_f = 10^{-2}$ or 10^{-3}, corresponding to $\beta = 2.32$ or 3.09, respectively).

As the calculated values of γ_g are not significantly higher than 1.0, it was decided to divide the computed values of γ_R and γ_E by γ_g, so that γ_g can be always taken equal to 1.0. Accordingly, only the load factor γ_E applied on the load-effect of the nominal seismic action, and the γ_R factor on the design flexural capacity, M_{Rd}, are given. The latter is like an element behaviour factor applied not only on the load-effect of the nominal seismic action, but also on that of gravity loads. Results for T-beams in positive bending are obtained for counteracting gravity loads (the - sign in Eq.(6)). As shown in Figs. 1 and 2, both factors increase with the ratio of bending moments due to nominal seismic action and gravity load, tending to constant limiting values for high values of this latter ratio. The γ_E-factors are higher than the ratio of the expected (mean) spectral accelerations in 50 yrs, to that of the nominal seismic action used for design, but still less than 1.0. It depends, though, on the type of element (beam vs. column). The behaviour factor, γ_R, depends stronly on the type of element and less on the ratio of load-effects. Its values for the different elements are lower than the corresponding (mean) rotation ductility supplies, but higher than the usual overall behaviour factors suggested by modern seismic codes.

The present results show that a uniform member (notional) failure probability cannot be achieved through constant partial safety factors on load and resistances. For values of the ratio of load-effects due to the nominal seismic action and gravity loads up to 10., partial safety factors should be taken as functions of this ratio, to achieve a constant notional failure probability of the members. More importantly, and regardless of the value of the load-effects ratio, partial safety factors for seismic design should be different for columns and for beams in negative (tension in the flange) or in positive bending. This latter difference is large, and is due to the inherently different available ductility factors of these members. Unfortunately, modern seismic codes, which use a global behaviour factor for the entire structure, do not account for these important effects.

ACKNOWLEDGEMENT

This work was sponsored by the Hellenic General Secretariat for Research and Technology.

REFERENCES

Ellingwood, B., T. Galambos, J., MacCoregor, & C.A. Cornell 1980. Probability based load factors for American National Standard A58. NBS Sp. Publ. 577. U.S. Dept. of Commerce, Nat. Bureau of Standards, Washington D.C.

Economou, S., & M.N. Fardis 1990. Seismic hazard analysis on spectral accelerations including statistical uncertainty: Application to sites in Greece. Proc. 9ECEE, Moscow.

Economou, S. & M.N. Fardis 1992. Probabilistic modeling and simulation of maximum elastic response spectra in a structure's lifetime. Proc. 1st Greek Conf. on Earthq. Engineering and Eng. Seismology, Nat.

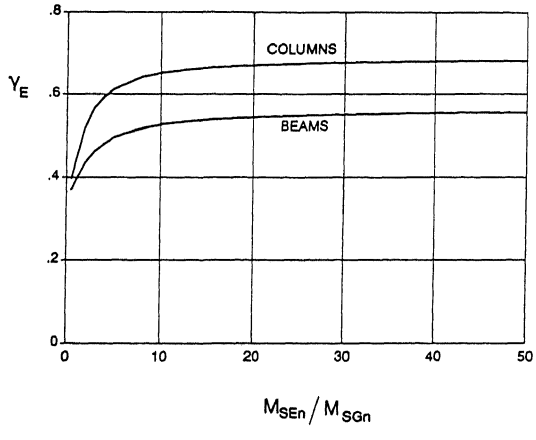

Figure 1. Load factor γ_E on nominal seismic action load-effect, M_{SGn}, for $\gamma_g = 1$ and $P_f = 10^{-1}$.

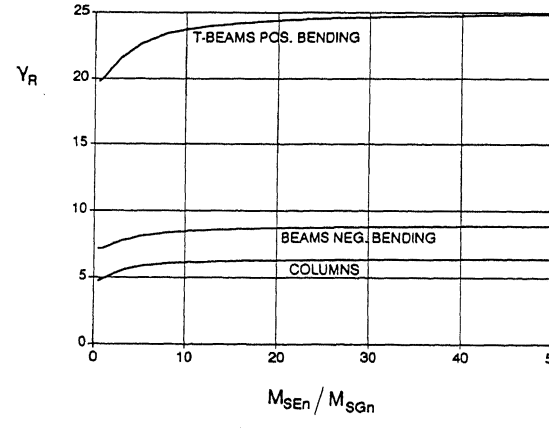

Figure 2. Member "behaviour factor" γ_R on M_{Rd}, for $\gamma_g = 1$ and $P_f = 10^{-1}$.

Chamber of Greece, Athens.
Darwin, D. & C.K. Nmai 1986. Energy dissipation in RC beams under cyclic load. ASCE J. Struct. Division, 112, 8: 1829-1846.
Fardis, M.N. 1991. Member-type models for the nonlinear seismic response analysis of reinforced concrete structures. In Experimental and Numerical Methods in Earthquake Engineering, J. Donea & P.M. Jones, Editors, Kluwer Academic Publishers, Dordrecht: 247-280.
Keintzel, E. 1986. Criteria and hypotheses for the determination of curvature ductility factors in RC structures: A comparative study. Proc. 8ECEE, Lisbon. 8.1: 71-78.
Lybas, J.M. & M.A. Sozen 1977. Effect of beam strength and stiffness on dynamic behaviour of reinforced concrete coupled walls. Res. Report, Dept. of Civil Eng., Univ. of Illinois at Urbana-Champaign.
McGuire, R.K. 1974. Seismic response risk analysis, incorporating peak response regressions on earthquake Magnitude and distance. Res. Report R74-51, Dept. of Civil Eng., M.I.T. Cambridge, Mass.
Park, Y.J. & A.M.-S. Ang 1985. Mechanistic seismic damage model of reinforced concrete. ASCE J. Struct. Division, 111, 4: 722-739.
Tassios, T.P. 1988a. Justification Note No. 6 - Background Documents for Eurocode No. 8: Part 1. V.2 - Design Rules: 24-49.
Tassios, T.P. 1988b. Justification Note No. 4 - Background Documents for Eurocode No. 8: Part 1. V.2 - Design Rules: 15-16.
Trifunac, M.D. 1976. Preliminary analysis of the peaks of strong earthquake ground motion-Dependence of peaks on earthquake Magnitude, epicentral distance and recording site conditions. Bull. Seism. Society of America. 66(1): 189-219.
Trifunac, M.D. & J.G. Anderson 1977. Preliminary empirical models for scaling absolute acceleration spectra. Rep. 77-03. Dept. of Civil Eng., Un. of Southern California, Los Angeles.