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TARGET RELIABILITIES FOR MINIMUM LIFE-CYCLE COST DESIGN: APPLICATION TO A CLASS OF R.C. FRAME-WALL BUILDINGS

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ABSTRACT

A methodology for the development of optimal target reliabilities for earthquake-resistant design of a class of ductile reinforced concrete (R.C.) frame-wall structures is presented. Optimal design reliabilities for damage control and life safety limit states are determined through the minimization of expected life-cycle costs, with the respective limit states quantitatively defined according to the Park-Ang damage index. The method is illustrated for a specific class of 5-story R.C. ductile frame-wall building structures located in Tokyo and designed according to the Japanese design code (BSL). The proposed methodology and application involve the evaluation of the initial cost and expected life-cycle damage cost as functions of the underlying risks or limit state probabilities under all likely ground motion intensities. The optimal level of safety is then determined on the basis of a trade-off between the expected damage cost from future earthquakes relative to the initial building cost, performed on the basis of minimum expected life-cycle cost. This constitutes a systematic procedure to determine the optimal target reliability from which cost effective criteria for aseismic design may be developed.

KEYWORDS

Risk and impact assessment; damage assessment; life-cycle cost-benefit analysis; earthquake resistant design; design criteria; shear walls; reinforced concrete buildings.

INTRODUCTION

Earthquake-resistant design and seismic safety assessment should explicitly consider the underlying randomness and uncertainties in the earthquake loading and structural capacities and ought to be formulated in the context of reliability. Moreover, since it is impractical to avoid damage under all likely earthquake loadings, the development of earthquake-resistant design criteria must include the possibility of damage and the need for repairs over the life of the structure. In this regard, optimal design criteria is meaningful only in terms of the expected total life-cycle cost including initial cost and repair and other damage costs. Although these issues are generally recognized and have been addressed by some researchers in the past (*e.g.*, Liu *et al.*, 1972, 1976; Rosenblueth, 1976) who have presented mathematical formulations of aseismic design on the basis of minimum life-cycle cost, the significance of economic factors has not been integrated explicitly with technical issues in the development of criteria for aseismic structural design. Here, the previous works are expanded to formulate a comprehensive approach for the development of cost-effective criteria for design which is then applied to a specific class of R.C. frame-wall buildings in Japan.

The proposed methodology can be summarized as follows. First, a set of model buildings is designed for different levels of reliability or performance following the procedure of an existing design code, *e.g.*, designs obtained with the UBC code or the current Japanese design code (BSL) for several base shear coefficients. From these designs a relation between the initial cost of the structures and the corresponding reliability under all possible earthquake loadings is established. For each design, the expected total cost of structural damage, including repair cost and other losses, is also formulated as a function of the reliability or probability of failure under all likely earthquake loadings and expressed in a common basis with the initial cost, *i.e.*, expressed in terms of present worth. A trade-off between the initial cost of the structure and the damage costs is then performed to determine the target reliability that minimizes the expected total life-cycle cost.

Formulation of the initial and expected damage cost functions, and a brief description of the required damage model and structural damage and reliability assessments are presented below, followed by an illustrative application to a class of 5-story R.C. frame-wall buildings located in Tokyo.

FORMULATION OF COST FUNCTIONS

Briefly, the expected life-cycle cost function can be expressed as,

$$E[C_t] = C_i + E[C_d] \tag{1}$$

where, C_i is the initial cost of a structure, and C_d is the damage cost over the life of the structure composed of the following (with reference to buildings):

$$C_d = C_r + C_c + C_{ec} + C_{in} + C_f \tag{2}$$

in which: C_r = the repair or replacement cost of the structure; C_c = the loss of contents; C_{ec} = economic impact of structural damage; C_{in} = cost of injuries caused by structural damage; and C_f = cost of fatalities from structural damage or collapse.

For consistency, all costs in Eq. 2 must be expressed in a common basis, *e.g.*, in terms of the present worth, and damage under all likely future earthquakes must be considered. Whereas the initial cost is normally expressed in present value, damage costs are associated with structural damage or collapse associated with the possible occurrence of future earthquakes and, therefore, the present worth of the damage cost will necessarily depend on the occurrence time of future earthquakes. Assuming that the occurrence of possible future damaging earthquakes at the site constitutes a Poisson process and that earthquake occurrences and their intensity are statistically independent, the present worth of the expected damage cost from future earthquakes can be shown to be (Ang and De Leon, 1995):

$$E[C_d] = E[C_d'] \sum_{n=1}^{\infty} \left[\sum_{k=1}^{n} \frac{\Gamma(k, \alpha T)}{\Gamma(k, \nu T)} \left(\frac{\nu}{\alpha}\right)^k \right] \frac{(\nu T)^n}{n!} e^{\nu T} \tag{3}$$

where C_d' = current cost of damage; q = annual discount rate, ν = annual mean occurrence rate of significant earthquake intensities; $\alpha = \nu + \ln(1+q)$; and, $\Gamma(.)$ = incomplete gamma function.

The items in Eqs. 2 and 3 that comprise the total damage cost are expressed as functions of the structural damage level x; thus, for example, for item C_r' the expected cost would be

$$E[C_r'] = \int_{a_{min}}^{a_{max}} \int_0^{\infty} C_r'(x)\, f_{X|A_1}(x|a_1)\, f_{A_1}(a_1)\, dx\, da_1 \tag{4}$$

where: x = the damage level; A_1 = expected maximum ground acceleration conditional on the occurrence of an earthquake; $f_{X|A_1}(x|a_1)$ = the probability density function (PDF) of X conditional on $A_1 = a_1$; and $f_{A_1}(a_1)$ = probability denisity function of A_1. Structural damage, x, and its PDF can be calculated for a given earthquake, from which the damage costs, *e.g.*, C_r', are determined as a function of the damage level, x.

STRUCTURAL DAMAGE AND RELIABILITY ASSESSMENT

Extensive assessments of structural damage and reliability under various earthquake load intensities are obviously needed to compute the expected total life-cycle cost. Here, a well-established damage model (Park and Ang, 1985) is extended for R.C. shear walls and used together with Monte Carlo simulation to compute the desired limit state probabilities.

Damage of a reinforced concrete component, column, girder or shear-wall is defined by the index (Park and Ang, 1985)

$$D = \frac{\delta_m}{\delta_u} + \frac{\beta_0 E}{Q_y \delta_u} \tag{5}$$

where: D = member damage index; δ_m = maximum displacement; E = hysteretic energy dissipated; Q_y = yielding force; δ_u = displacement capacity; β_0 = constant.

Global damage is defined as a function of the damage of its constituent elements or components, particularly, the critical components. Denoting damage of a critical component or substructure by D_i, global damage is defined as:

$$(D_G > d) = \bigcup_i (D_i > d) \tag{6}$$

where: D_G = global damage; $\bigcup$ = union of events. Damage of a critical component or substructure is sometimes computed as a combination of the damage of several components such as, for example, a story damage that is computed as a weighted average of the damage of several story columns and shear walls.

To properly assess component damage under random seismic loads, the structure must be adequately modeled and analyzed to compute its response under simulated or recorded earthquake ground motions. Since the structural response under severe and moderate earthquake loads is nonlinear-hysteretic, the computation of the response statistics under random earthquake loads using appropriate random structural models and capacities, becomes an extremely complex task. Here, Monte Carlo simulation is used to compute the desired response statistics. In this regard, the computer program DRAIN-2DX has been modified to perform the desired Monte Carlo simulations, with the critical structural components modeled using the DRAIN-2DX beam-column element with trilinear elasto-plastic hysteresis. The earthquake ground motions used as input can be either actual earthquake records or samples of nonstationary filtered Gaussian processes with both frequency and amplitude modulation (Yeh and Wen, 1990).

To reduce computational costs and time, a relatively small number of simulations, of the order of a few hundreds, is performed with this program, and a joint probability distribution function (PDF) is fitted to the computed damage response statistics. This joint PDF of the relevant damage statistics is then used to compute the desired limit state probabilities using Eq. 6. Uncertainties in structural properties and capacities and in critical earthquake loading parameters are modeled as lognormal variables, and a lognormal distribution is chosen also for the damage response statistics. It has been verified that a lognormal PDF for a component damage obtained with a few hundred simulations fits well the simulation results obtained with a sample size of 10,000.

Damage Model. Models and equations to quantify the damage index parameters for R.C. columns and girders, *i.e.*, to quantify δ_u, Q_y and β_0, proposed by Park and Ang (1985) and Park, Ang and Wen (1987) are used in this study. For R.C. shear wall components, a set of empirical equations has been proposed in Kunnath, Reinhorn and Park (1990) to compute δ_u, β_0 and Q_y. The equation for β_0 proposed by Kunnath, Reinhorn and Park (1990) is retained here, whereas a different approach is used to compute the other two parameters of the damage index as explained below.

For slender shear walls a curvature ductility capacity, μ_ϕ, is computed on the basis of the flexure theory for reinforced concrete shear walls (Paulay and Priestley, 1992), considering a ultimate concrete crushing strength $\epsilon_u = 0.004$ and taking into account the strain-hardening of the tensile steel reinforcement. A displacement ductility capacity, μ_Δ, for each wall component is then computed extending the relationships between curvature and displacement ductility suggested by Paulay and Priestley (1992). The displacement capacity δ_u is then given by $\delta_u = \mu_\Delta \delta_{fy} + \delta_S$, where δ_{fy} = flexure yielding displacement and δ_S the shearing contribution to the total displacement. Relationships between displacement demands and the computed local maximum concrete compressive strain have been used by others, *e.g.*, by Wallace (1984) to propose a new design methodology for seismic design of slender R.C. shear-walls. For the frame-wall structures considered in this study, computation of the displacement capacity, δ_u, and of the yielding force, Q_y, for each structural wall component requires knowledge of the shear span for the wall, which is obtained on the basis of linear time-history response analyses. If more sophisticated structural elements are used, *e.g.*, the so-called fiber element, the concrete compressive strains and curvature ductilities can be computed during the analysis, which would permit a formulation of the damage parameter in terms of bending moments and curvature.

If the predicted failure mode for the wall is shear failure, the displacement capacity is computed using an empirical correlation between measured displacement capacities and computed wall deflections at cracking and yielding, established by the authors on the basis of 27 small-scale test results reported by Hirosawa (1975). The shear strength capacity of the wall for such predicted failure modes is computed on the basis of Eq. 4.6.2 in Hirosawa (1975).

ILLUSTRATIVE APPLICATION

The general approach described above is illustrated for a class of Japanese 5-story R.C. frame-wall building structures located in the city of Tokyo. A brief description of the model buildings and damage cost functions is presented followed by a summary of the results obtained.

Model Buildings. The building class investigated consists of 5-story R.C. ductile frame-wall structures such as the one illustrated in Fig. 1 and located in a soft soil site within the city of Tokyo. Each structure consists of 4 paralell frames similar to that shown in Fig. 1, and three 3-bay frames in the direction normal to that shown in Fig. 1. This study considers only the building response in the direction indicated in Fig. 1. Several model buildings have been designed for the service-level design earthquake specified in the Japanese code (BSL) for seismic coefficients C_0 ranging from 0.10 to 0.50, where C_0 is the standard base shear coefficient currently equal to 0.2 for the service-level design earthquake. It is noted that for the ultimate lateral strength limit state it is $C_0 = 1.0$, and that a structural reduction factor, D_S, that depends on the available ductility, is allowed by the code to reduce the code specified linear response spectrum. Values for D_S range from 0.25 to 0.55.

A finite element model, such as the one shown in Fig. 1, is constructed for each model building and used for the required structural damage and reliability assessments.

Expected Damage Cost Function. On the basis of reported damage repair cost data and of damage assessment analyses of buildings damaged under previous earthquakes in Japan, a regression relation was obtained between the computed median global damage index and the actual or estimated damage repair cost. This regression relation is shown in Fig. 2 for a median global damage less than 0.5. Data used for establishing this regression relation are taken from the Architectural Institute of Japan report on the Miyagi-ken-oki earthquake of 1978 (AIJ, 1980) and from the Public Works Research Insitute report on the same earthquake (PWRI, 1983) for buildings A to C; and from Hirosawa *et al.* (1985) and Hirosawa, Yamazaki and Tsukagoshi (1984), for buildings D and E, respectively. The 3 frames of building C and the critical frame of building D are shown in Fig. 1. It is noted that building C was demolished rather than repaired, but repair costs have been estimated for this building.

With these data, the damage repair cost component has been developed as follows:

$$C'_r = 0.57 C_i D_m; \qquad 0 \leq D_m \leq 0.5 \tag{7}$$

$$ = C_i; \qquad D_m > 0.5 \tag{8}$$

where D_m = computed median global damage for the structure, C_i = replacement cost of the original structure, with the limit of reparable damage for the buildings analyzed taken to be equal to $D_m = 0.5$.

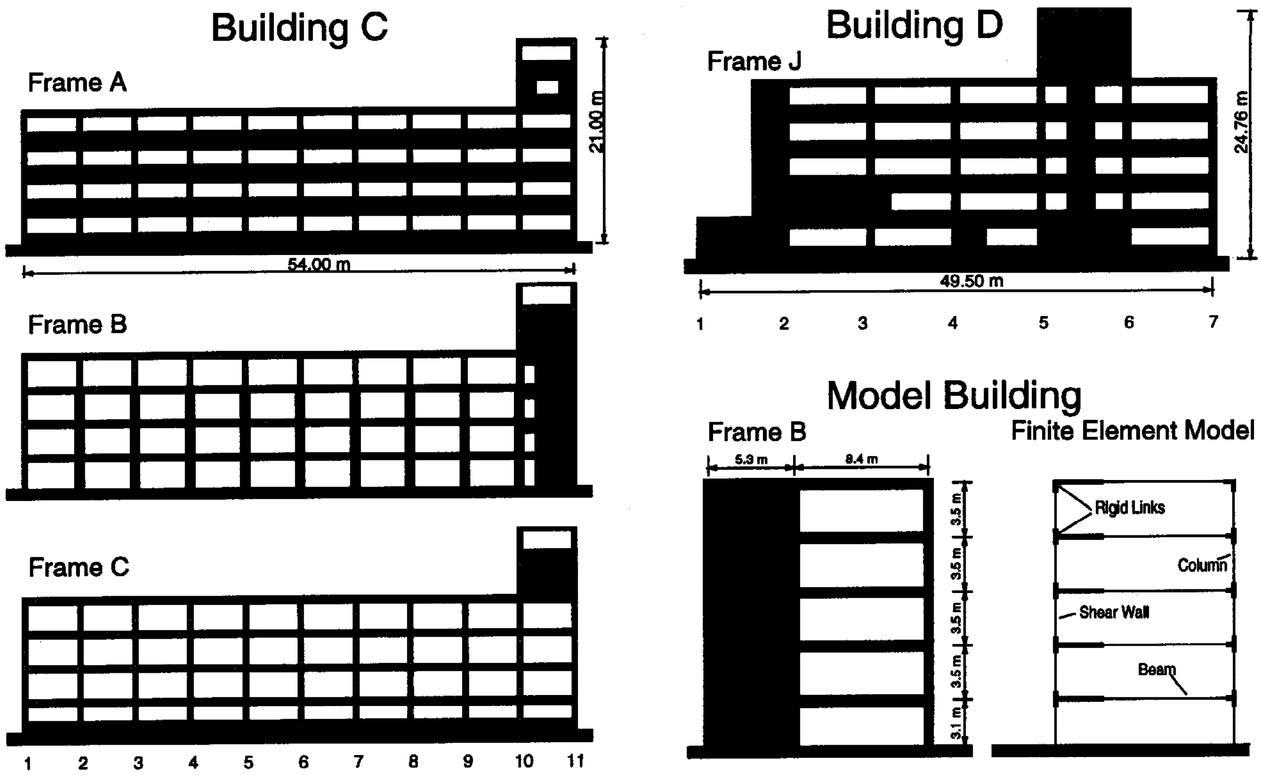

Fig. 1. Elevation views of frames for buildings C, D and the model buildings, and finite element model of model buildings

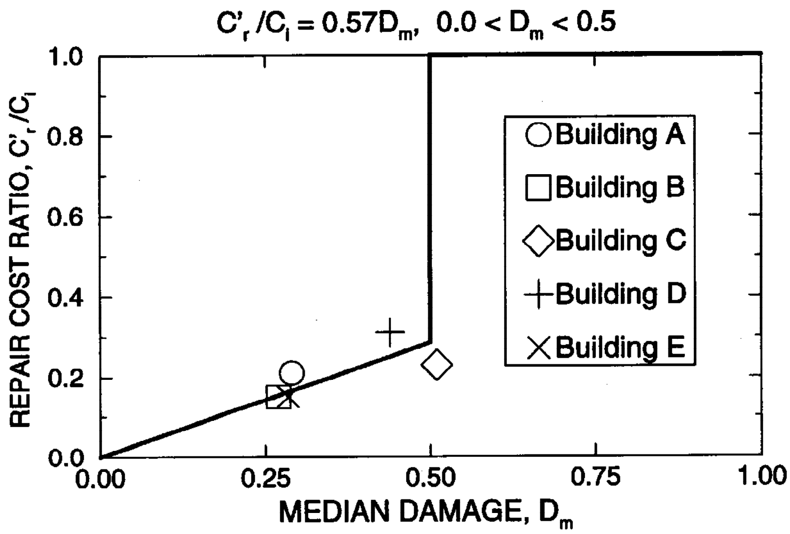

Fig. 2. Damage repair cost function

The total loss of contents is assumed to reach a maximum of 40% of the replacement cost and to vary linearly with D_m for intermediate damage as follows:

$$C'_c = 0.4C_iD_m; \qquad 0 \leq D_m \leq 0.5 \tag{9}$$

$$= 0.4C_i; \qquad D_m > 0.5 \tag{10}$$

The direct economic loss from loss of rental revenue is determined assuming that the loss of rental revenue if the building collapses or exceeds the limit of reparable damage is equal to 23% of the replacement cost of the building, and varies linearly with the median damage up to the limit of reparable damage; thus

$$\begin{aligned} C'_{ec} &= 0.23(2.0D_m)^2; \quad 0 \leq D_m \leq 0.5 && (11) \\ &= 0.23C_i; \quad D_m > 0.5 && (12) \end{aligned}$$

This cost function has been developed on the basis of the average rental fees per square meter per month for office buildings at the site, and assuming that one and half years will be needed to reconstruct the building.

The cost of fatalities is determined on the basis of the expected loss to the national GDP. The number of fatalities per unit floor area for $D_m \geq 1$, are estimated on the basis of the number of fatalities per collapsed building under the 1995 Kobe earthquake (Sakamoto and Indrawan, 1995) and assuming an average floor area of 100 m^2 for the collapsed buildings, whereas the expected loss to the GDP is estimated at 100×10^6 Yen per fatality. For intermediate values of damage the costs are made proportional to the 4th power of the damage index, thus:

$$\begin{aligned} C'_f &= 0.157\, A_F\, (D_m^4); \quad 0 \leq D_m \leq 1.0 && (13) \\ &= 0.157\, A_F; \quad D_m > 1.0 && (14) \end{aligned}$$

where A_F = building floor area in m^2. For the cost of injuries it is assumed that for $D_m \geq 1$, ten percent of all injuries are disabling injuries, the average cost of non-disabling injuries is 5×10^6 Yen, the loss from a disabling injury is equal to the loss from a fatality, that the number of injuries per unit floor area is one and a half times that of fatalities and that the cost of injuries is proportional to the square of the damage index for intermediate damage levels, thus:

$$\begin{aligned} C'_{in} &= 0.0341\, A_F\, (D_m^2); \quad 0 \leq D_m \leq 1.0 && (15) \\ &= 0.0341\, A_F; \quad D_m > 1.0 && (16) \end{aligned}$$

All costs indicated above are expressed in terms of millions of Yen.

Optimal Reliabilities and Base Shear Coefficient. Expected total life-cycle costs for 9 model buildings are shown in Fig. 4 as a function of the computed risk, *i.e.*, the probability that a specified damage threshold be exceeded over the 50-year lifespan of the structure, for q = 3% and for the two seismic hazard curves shown in Fig. 3. The seismic hazard curve labelled I is taken from Ishikawa and Kameda (1988) and uses an attenuation equation for soil type 4 (soft soils) as defined in the 1980 Specifications for Highway Bridges of the Japan Road Association, whereas the seismic hazard curve labelled II is computed using the methodology and data described by Kameda (1987) and Kameda *et al.* (1988), which makes use of an attenuation equation for rock sites togehter with site amplification effects. It is noted that the method proposed by Kameda (1987) to simulate earthquake ground motions differs from the one used here for the damage and reliability assessments.

The optimal base shear coefficient obtained on the basis of seismic hazard I is $C_0^* = 0.2$ which is equal to the one currently used, whereas the optimal base shear coefficient computed on the basis of seismic hazard II is between 0.20 and 0.25. The computed optimal probabilities of ($D_G > 1$) are, approximately, 1.2×10^{-3} for both hazard functions. Although the computed optimal reliabilities under both seismic hazard functions are similar, seismic hazard II results in greater expected damage costs and a slightly greater optimal base shear coefficient because it implies return periods for intermediate to large peak ground accelerations smaller than those implicit in seismic hazard I.

Shown in Fig. 5 are the contributions of the initial cost and various damage cost components to the expected total cost under seismic hazard I. The results indicate that the repair/replacement cost and loss of contents contribute the most to the expected total life-cycle cost, and that the expected damage cost can be a significant part of the expected total life-cycle cost.

The effect of the discount rate, q, on the expected total cost, optimal seismic coefficient and reliability is shown in Fig. 6 under seismic hazard I. As the discount rate increases, the expected total life-cycle cost decreases and a more pronounced optimum at $C_0 = 0.20$ is observed. Although results for discount rates greater than 5% are not computed, it appears possible to conclude that the optimal base shear coefficient also depends on the economic structure, with greater significance being placed in the expected life-cyle damage cost for the smaller discount rates, which may lead to greater optimal base coefficients for the lower discount rates.

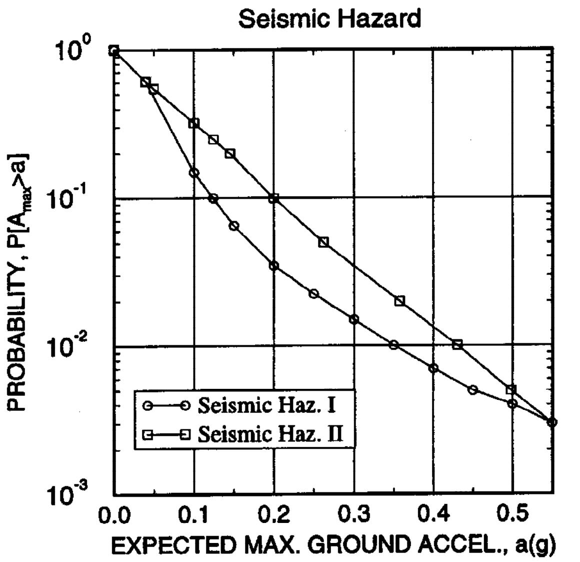

Fig. 3. Seismic hazard functions at the site

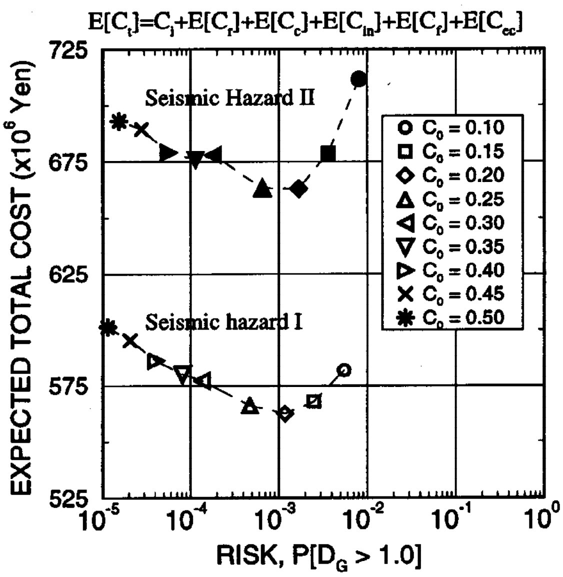

Fig. 4. Expected total life-cycle cost and risk for two hazard functions

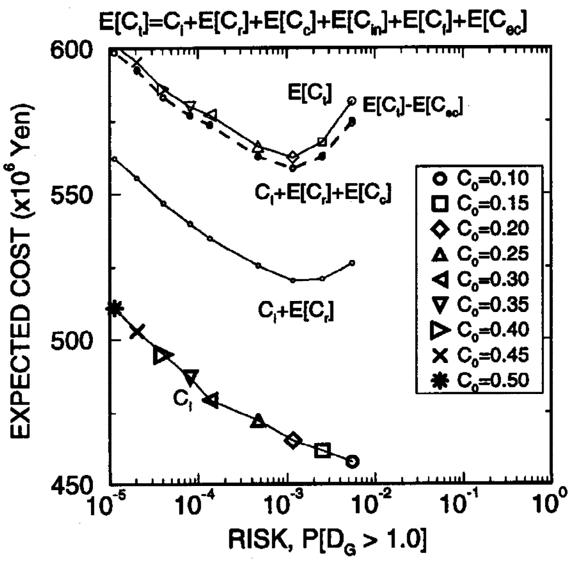

Fig. 5. Contribution of various costs to the expected total cost

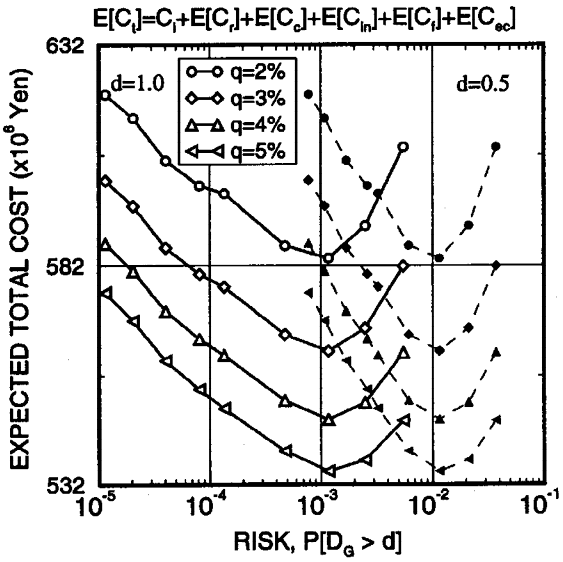

Fig. 6. Effect of discount rate on risk, cost and and optimal base shear coefficients

CONCLUSIONS

A methodology for the systematic development of cost-effective design criteria for earthquake-resistant design that integrates the relevant technical factors with socio-economic factors is presented and illustrated for a class of 5-story R.C. frame-wall buildings. Optimal reliabilities are obtained for quantitatively defined limit states by minimizing the expected total life-cycle cost. The type of results that can be obtained may be useful and of significance in the development of performance based earthquake-resistant design criteria.

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