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A methodology for value based seismic design of structures by the endurance time method

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Abstract. A new design methodology based on the total value of structures is introduced. This methodology, namely Value Based Design of structures (VBD), uses the advantages of Endurance Time (ET) method. While prescriptive and earlier generations of performance based design approaches commonly try to find structures with the least initial cost, a design approach to directly incorporate the concept of value in design procedure has been formulated here. Reduced computational effort in ET analysis provides the prerequisites to practical use of optimization algorithms in seismic design. A genetic algorithm is used with the objective of minimizing total cost of the building during its lifespan. ET method is used to estimate the structural responses of each candidate design to probable earthquakes and the expected costs of earthquake consequences are calculated using Life Cycle Cost Analysis (LCCA). A prototype steel frame is optimally designed according to a prescriptive, performance based and the proposed value based design method. Then, their seismic performance and expected cost components are investigated. The results provide a pathway towards practical value based design and show that conformance to design code requirements or performance objectives does not assure achieving the best design regarding the overall design values.

1. Introduction

In recent decades, large economic losses following earthquakes have revealed the need for improved design criteria intended to reduce damages and economic impacts to an acceptable level along with life protection. The prescriptive and also earlier generations of performance based seismic design approaches try to design structures satisfying minimal requirements under seismic actions in a number of intensity levels and a design having lower initial cost is commonly preferred. Such approaches will not necessarily result in an economical design with lower total cost in lifetime of the structure. Thus, Life Cycle Cost Analysis (LCCA) has been applied in construction industry to account for economic concerns in decision making procedures. The expected costs caused by future earthquakes during the design life of a structure can be estimated using LCCA. This analysis, implemented in an optimization algorithm, can be used to find a design with the least total cost. Basically, this analysis can provide a baseline to incorporate technical, economic, and social or any other intended measures in design procedure. By using this method, the expected total cost of a structure, including the initial cost

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and losses caused by probable earthquakes, during its lifespan can be considered as the main measure for the priority of design alternatives. Here, LCCA is used to determine the total cost of a structure to be used in optimum design procedure. Readily introduced value based design can provide a wider description of design target by defining the earthquake consequences such as structural damages, loss of contents, losses due to downtime, human injuries, and fatalities in the form of quantifiable parameters. In this way, it is expected that the resultant design will perform with desired post-earthquake capabilities with manageable disruption.

LCCA demands the calculation of cost components associated with the performance of the structure in multiple hazard levels [1]. In order to have a reliable seismic performance assessment of a structural system to be used in the LCCA methodology, response-history based incremental analyses and realistic numerical models of the structure should be used. However, the huge computational demand required in these procedures and sophistications involved may make optimization algorithms impractical due to the repetitive nature of these algorithms, or the simplifications used will decrease the reliability of the results. In this research study, Endurance Time (ET) method, as a dynamic procedure requiring reasonably reduced computational effort, is applied to estimate the performance of the structure in all levels of hazard intensity [2]. The main procedure in ET method is to analyze structures subjected to predesigned intensifying accelerograms and assess their performance based on structural responses at different excitation levels. Using this method, performance of the structure can be monitored in a full range of hazard intensities by each single response-history analysis instead of progressively scaled up ground motions in IDA. Thus, the required huge computational demand of incremental dynamic analyses is considerably reduced while maintaining the major advantages of it, i.e. accuracy and insensitivity to model complexity [3]. The idea of using ET analysis results to calculate the simple expected cost was introduced by Basim and Estekanchi [4]. This potential capability will be used here to extend the application of detailed loss reduction metrics in a practical design procedure. This can pave the way for practical value based seismic design of structures.

Mirzaee et al. [5] and Hariri-Ardebili et al. [6] have studied the applications of the ET method in performance assessment of structures. Correlating the dynamic characteristics of ET intensifying excitations with those of ground motions at various hazard levels has resulted in reasonably accurate estimates of expected seismic responses at various excitation intensities through ET analysis [7].

In order to demonstrate the method, a five-story and three-bay steel special moment frame is optimally designed based on three distinct philosophies: first, according to Iranian National Building Code (INBC) as a prescriptive design code, which is almost identical to the ANSI/AISC360 [8] LRFD design recommendations; second, according to FEMA-350 [9] limitations as performance based design criteria; and third, using the introduced value based method to have the minimum total cost during its lifetime, which is assumed 50 years. A cost model appropriate for the studied building is defined and used to quantify the consequences of probable earthquakes. Although the proposed methodology is general and can be used for any type of constructions, the used cost model is defined for this specific case study of steel frame based on judgmental assumptions and more research is needed to provide generalized models for other types of buildings. Seismic performance and expected cost components of the resultant prescriptive, performance based, and value based designs of the frame are investigated and discussed.

2. Background

Although significant progresses have been made in the last two decades in the area of earthquake engineering, currently, most of the seismic design codes belong to the category of the prescriptive design codes, in which a number of limit state checks are recommended to provide safety. Indeed, prescriptive building codes aim at ensuring adequate strength of structural members and overall structural strength and, hence, they do not provide warrantable levels of building life cycle performance [1]. Thus, design codes are migrating from prescriptive procedures intended to preserve life safety to reliability based design methodologies and most of them have attempted to advance their design criteria towards new generations of Performance Based Design (PBD) of structures. In performance based earthquake engineering, the performance of the building in its lifetime is inspected in order to ensure reliable and predictable seismic performance. Several guidelines on this concept have been introduced over the last decade for assessment and rehabilitation of existing buildings and analysis and design of new ones. FEMA-350 [9] provides a probability based guideline for performance based design of new steel moment resisting frames considering uncertainties in seismic hazard and structural analyses.

In performance based design, after selecting the performance objectives and developing a preliminary design, seismic response of the design is evaluated and, afterwards, the design is revised until the acceptance criteria for all intended performance objectives are met. More time-consuming analysis procedures are employed in PBD to estimate the non-linear structural responses in different levels of excitation. Optimization methods have been effectively used for PBD to achieve optimal designs with acceptable performance while the structural performances and also structural weight are treated as objectives or constraints of the optimization problem [10]. Among many others, Pan et al. [11] used a constraint approach to incorporate several design requirements into a multi-objective optimization problem and Liu et al. [12] formulated the performance based design procedure subjected to uncertainties as a multi-objective optimization problem and used genetic algorithm to provide a set of Pareto-optimal designs.

Recently, researchers have tried to introduce financial concerns in structural design area to reduce the amount of economic losses caused by earthquakes and hurricanes. As a result, LCCA has become an important part of structural engineering to assess the performance of the structures during their lifespan in economic terms. As one of the impressive works in this area, Wen and Kang [13] formulated long-term benefit versus cost considerations for evaluation of the expected life cycle cost of an engineering system under multiple hazards. Later, Liu et al. [14] used a multi-objective genetic optimization algorithm to automate the design procedure and find optimal design alternatives with respect to three objectives. They used static pushover analyses to assess the performance of steel frame design alternatives. Takahashi et al. [15] used a renewal model for the occurrence of earthquakes in a seismic source to formulate the expected life cycle cost of design alternatives and applied the methodology to an actual office building as a decision problem. Liu et al. [16] used a multi-objective optimization method to automate the performance based seismic design of steel frame structures considering the seismic risk in terms of maximum inter-story drift. Fragiadakis et al. [17] compared single-objective optimal design with minimum initial weight and a performance based two-objective optimum design of a steel moment resisting frame; meanwhile, they presented a framework to obtain a Pareto front of the design alternatives. Mitropoulou et al. [18] investigated the effect of the behavior factor in the design of reinforced concrete buildings under earthquake loading in terms of safety and economy by comparing initial and damage cost components of each design. Mitropoulou et al. [1] explored the effect of some analysis characteristics on the life cycle cost analysis of reinforced concrete structures. Jennings [19] used a multi-objective optimization algorithm with socioeconomic and engineering objectives to identify optimal retrofit plans for wood-frame building stock of a community in order to improve community resiliency. Four contributors of losses were considered in this reference: initial cost, economic loss, number of morbidities, and recovery time and some other complementary measures were used to account for the loss in quality of life for the population.

3. Endurance Time method (ET)

ET excitation functions are in the form of artificial accelerograms generated in such a way that response spectrum of any time window of them from zero to a particular time matches a template spectrum with a scale factor which is an increasing function of time. Numerical optimization procedures have been used to achieve this interesting characteristic [20]. Various sets of ET acceleration functions have been produced with different template response spectra and are publicly available through the website of ET method [21]. A typical ET accelerogram used in this work, ETA40h, is depicted in Figure 1. These records are optimized to fit average response spectrum of 7 records (longitudinal accelerograms) used in FEMA-440 for soil type (C) as template spectrum.

As it can be verified in Figure 2, the response spectrum of a window from $t_{\mathrm{ET}} = 0$ to $t_{\mathrm{ET}} = 10$ sec of the used accelerogram matches the template spectrum. Furthermore, the produced response spectra by other time windows of the record also match the template spectrum with a scale factor, providing a correlation between analysis time and induced spectral intensity.

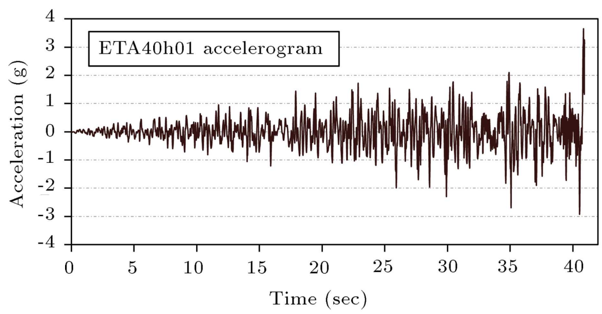

Figure 1. Acceleration function for ETA40h01.

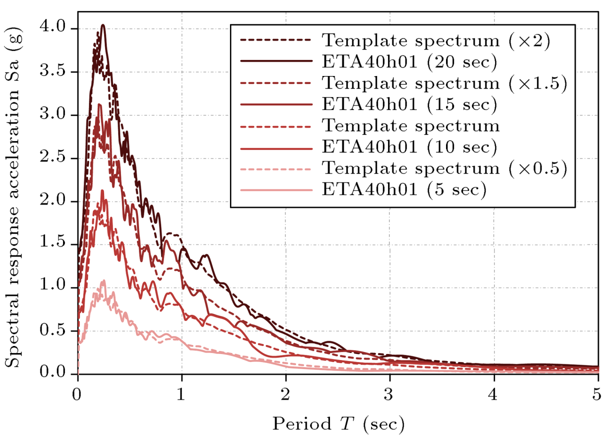

Figure 2. Acceleration response spectra for ETA40h01 at different times of excitation.

Therefore, each ET analysis time can be attributed to a particular seismic intensity and analysis results can be more effectively presented by substituting equivalent hazard return period for ET analysis time considering the fact that hazard levels are well presented by acceleration response spectra in the current codes [22]. This can provide an appropriate baseline to calculate expected damages and costs.

In a work by Mirzaee et al. [5], application of the ET method in performance based design was studied introducing "Performance Curve" and "Target Curve", which respectively expressed the seismic performance of a structure along various seismic intensities and their limiting values according to code recommendations. Hazard return periods corresponding to any particular time in ET analysis are calculated here by matching the response spectra obtained from the ET accelerogram at different times and response spectra defined for Tehran at different hazard levels. The procedure is based on the coincidence of response spectra at effective periods, i.e. from 0.2 to 1.5 times of structure's fundamental period of vibration. The results show that substitution of the return period or annual probability of exceedance for time in ET analysis and performance curves will make the results more explicit and also increases the usefulness of these curves in calculating expected costs. The variation of the return period with the structural period and analysis time in ET analysis is illustrated in Figures 3 and 4. This correlation provides the corresponding ET time for each hazard level for a specific structure.

In the following sections, ET curve is used to assess the performance of the obtained design for the five-story structure compared with the target curve. In these figures, ET analysis time has been mapped into return period on horizontal axis and moving average is applied to smooth ET results for inter-story drift envelope curve. The performance of the structure at various hazard return periods can be verified on these figures. The code limitations on structural responses at various hazard levels can also be checked as the target curve. This is one of the advantages of the ET method that the performance of a structure in all hazard levels can be properly depicted in an easy to read figure.

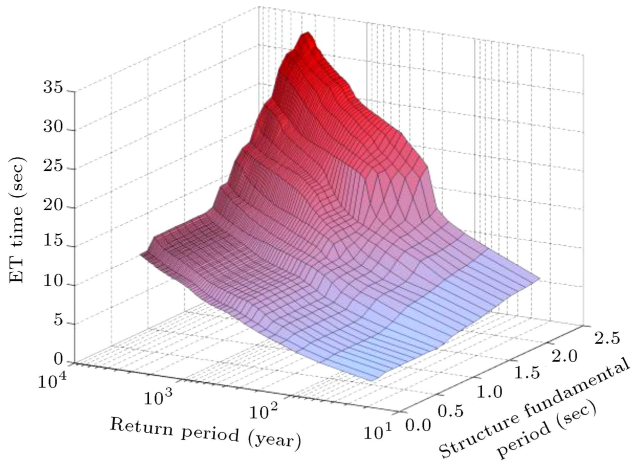

Figure 3. Return period vs. structural period and ET analysis time.

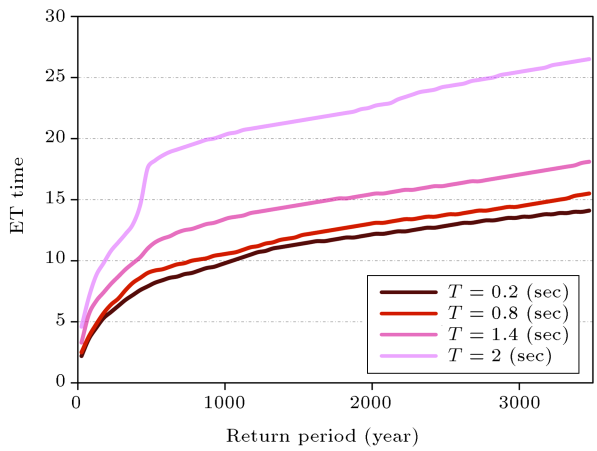

Figure 4. Equivalent ET analysis time vs. hazard return period for different structural periods.

4. Prescriptive seismic design

Commonly, in prescriptive seismic design procedures, structures are checked in one or two deterministically expressed limit states (i.e., ultimate strength and serviceability). In these procedures, the elastic base shear is reduced by a behavior factor (R) to incorporate the inelastic deformation capacity of the structure.

At first, a five-story structure is optimally designed according to the Iranian National Building Code (INBC) as a prescriptive design code, which is almost identical to the ANSI/AISC360-10 LRFD design recommendations. The prototype structure is a five-story and three-bay special moment resisting steel frame. All supports are fixed and the joints are all rigid. The beams and columns are selected among seismically compact standard W profiles according to Table 1. The geometry of this model can be found in Figure 5. Loading is set according to Iranian National Building Code, Section 6. The structural steel material has yielding stress, $F_y = 235.36$ MPa, and elastic modulus, $E = 200$ GPa. The strong-column/weak-beam design requirement has been considered in design of the structure. According to Iranian Seismic Design Code, seismic loading base shear is determined upon design response spectrum of the 475-year return period hazard level and the elastic base shear is reduced by the behavior factor to allow the structure to absorb energy through inelastic deformations. Demand/capacity ratios for an optimum prescriptive design are depicted in Figure 5. As can be seen in this figure, other

Table 1. List of alternative section properties for beams and columns.

Beams	$W18$ $\times35$	$W18$ $\times40$	$W18$ $\times46$	$W18$ $\times50$	$W18$ $\times55$	$W18$ $\times60$	$W18$ $\times65$	$W18$ $\times71$						
Side columns	$W10$ $\times39$	$W10$ $\times45$	$W10$ $\times54$	$W10$ $\times60$	$W10$ $\times68$	$W10$ $\times77$	$W10$ $\times88$	$W10$ $\times100$	$W10$ $\times112$					
Inner columns	$W12$ $\times79$	$W12$ $\times87$	$W12$ $\times96$	$W12$ $\times106$	$W12$ $\times120$	$W12$ $\times136$	$W12$ $\times152$	$W12$ $\times170$	$W12$ $\times190$	$W12$ $\times210$	$W12$ $\times230$	$W12$ $\times252$	$W12$ $\times279$	$W12$ $\times305$

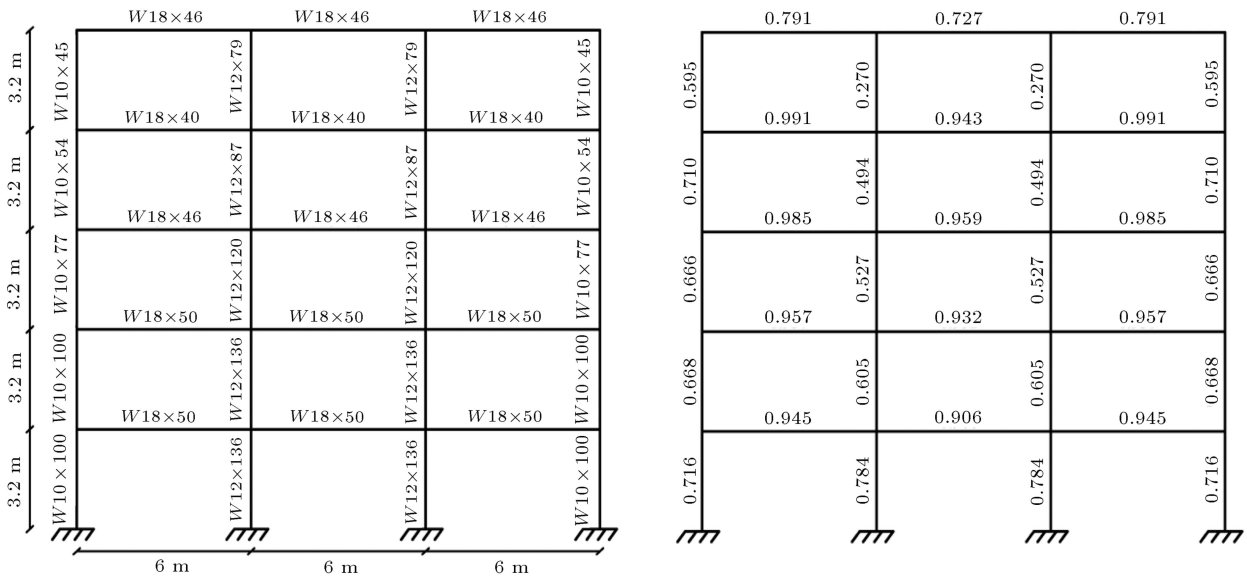

Figure 5. Schematics of steel frames under investigation and demand/capacity ratios according to prescriptive design criteria.

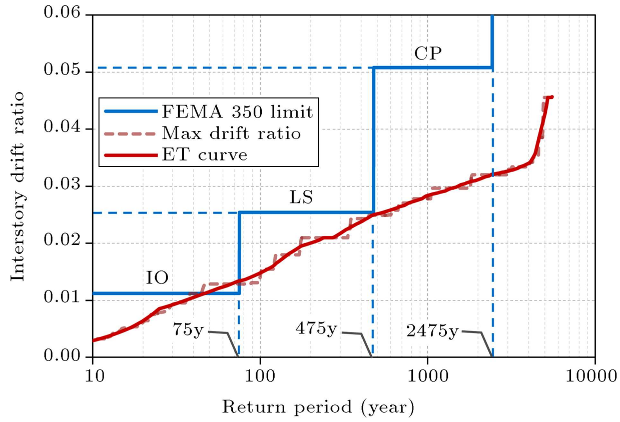

Figure 6. Performance curve (ET curve) for the prescriptive design.

limitations such as drift limits or strong-column/weak-beam limitation have dominated the design in some elements.

In Figure 6, the seismic performance of the prescriptive design has been investigated according to FEMA-350 limitations on inter-story drift ratios by the ET method. Performance curve (ET curve) and also target curve for this structure are compared in this figure. FEMA-350 criteria for performance assessment are explained in the next section. It can be verified that the structure has violated IO level limitation but has a proper performance in LS and CP levels.

5. Performance based design

Prescriptive design procedures do not assure reliable performance of the structure in multiple hazard levels during its lifespan since these procedures merely intend to keep the ultimate strength of structural members at an acceptable level. Compared to these procedures, in Performance Based Design (PBD), a more general structural design philosophy is used in which the design criteria are expressed in terms of achieving multiple performance requirements when the structure is subjected to various seismic hazard levels. In most of the currant performance based design criteria, the objective performance of an ordinary building is usually defined to resist a significant accidental earthquake without structural damage, allow repairable structural damage against a rare major earthquake, and resist the maximum credible earthquake without collapse [11]. In order to assess the performance of structures, various response estimation methods, such as push-over analysis which is widely used in this area, have been used by researchers and also engineers; however, time history analysis is so far believed to be the most accurate methodology for evaluating structural performance. Optimization methods in design procedures have also been used to achieve safe and economical designs which can satisfy the performance based measures. An optimum performance based design methodology is introduced in a work by Estekanchi and Basim [3] utilizing ET method as analysis tool.

The performance based design measures implemented in this work are based on FEMA-350 [9]. FEMA-350 has provided a guideline for performance based design of new steel moment resisting frames. In these criteria, a probability based approach is used to explicitly consider the ground motion variability and the uncertainty in the structural analysis. Two discrete structural performance levels, Collapse Prevention (CP) and Immediate Occupancy (IO), are considered in FEMA-350. Limitations on inter-story drifts and forces in various elements, especially in columns, are defined in these criteria for each of these performance levels. Other structural performance levels can be determined on a project-specific basis by interpolation or extrapolation from the criteria provided for the two performance levels. For the purpose of this work, Life Safety (LS) performance level has been defined by interpolating the IO and CP levels. In LS level, the structure experiences significant damages resulted by the hazard, although some margin remains against either partial or total collapse. The performance objective in this study assuming "seismic use group I" for the prototype special moment frame structure is defined as achieving IO, LS, and CP performance levels in the case of ground motion levels of 50%, 10%, and 2% probability of being exceeded in 50 years, respectively.

Many uncertainties are involved in behavior and response of a building ranging from uncertainties in seismic hazard due to the attenuation laws employed and record to record variability to uncertainties in structural modeling due to simplifications and assumptions used in the numerical analysis [12]. To account for these uncertainties, FEMA-350 uses a reliability based probabilistic approach to define performance measures that explicitly acknowledge these inherent uncertainties. These uncertainties are expressed in terms of a confidence level. A high level of confidence means that the building will very likely be capable of meeting the desired performance. Considering a minimum confidence level of 90% for IO and CP performance levels, the upper-bound limits for the calculated inter-story drift demand obtained from structural analysis would be 0.0114 and 0.0508 and interpolation will result in an upper bound of 0.0254 for LS level.

OpenSees [23] is used to perform structural response analyses. Concentrated plastic hinges by zero-length rotational springs with elastic beam-column elements are used to model the nonlinear behavior of elements. Plastic regions follow a bilinear hysteretic response based on the Modified Ibarra Krawinkler Deterioration Model [24,25]. To represent shear distortions in the panel zones, they are modeled as rectangles composed of eight very stiff elastic beam-column elements with one zero-length rotational spring in the corner based on the approach of Gupta and Krawinkler [26]. P-Delta Coordinate Transformation object embedded in the platform is used to consider the second-order effects.

A single-objective optimization problem is defined to find a design having the minimum initial steel material weight as optimization objective. The limitations on inter-story drift demand and axial compressive load on columns and also strong-column/weak-beam criterion according to FEMA-350 recommendations are formulated as optimization constrains. The design variables are the steel section sizes selected among standard W sections. As indicated in FEMA-350, structures should, as a minimum, be designed in accordance with the applicable provisions of the prevailing building codes, such as specifications of AISC360 [8] and AISC Seismic [27]. Thus, the AISC360 requirements and FEMA-350 acceptance criteria are implemented as initial design constraints. Optimum design sections have been determined using GA algorithm adopted for performance based design purposes using ET method introduced in a work by Estekanchi and Basim [3]. The optimum design sections can be found in Figure 7. A comparison between performance of the designed frame and the limiting curve according to FEMA-350 in various seismic intensities can be performed using the

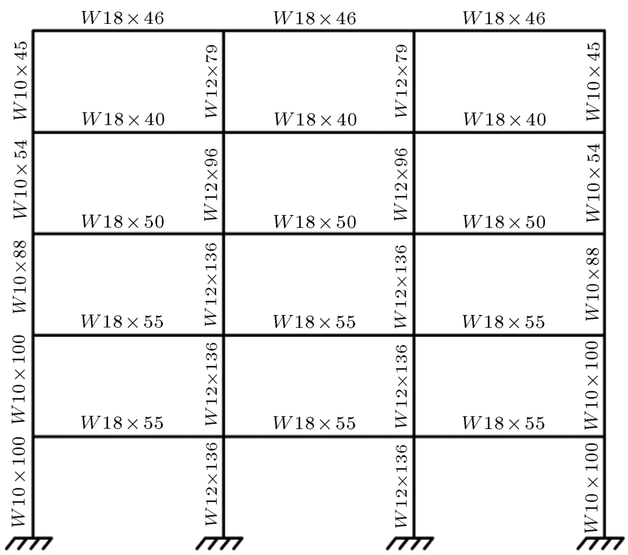

Figure 7. Performance based design sections of the frame.

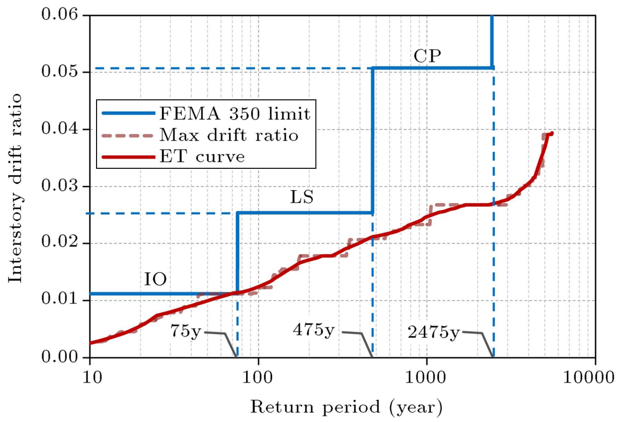

Figure 8. Performance curve (ET curve) for the performance based design.

ET curve presented in Figure 8. As could be expected, the optimum design meets the limitations (i.e., code requirements) with the least margins.

6. Value based design

While value can be defined and considered in its broad sense for design purposes, for clarity of explanation, in this research, we consider the structure that is more economical to construct and maintain to be the most valued. As it will be demonstrated in this section, ET analysis provides a proper baseline to perform economical analyses on design alternatives with acceptable computational cost. Initial construction cost and expected seismic damage cost throughout the lifetime of the structure are usually the two most important parameters for decision making [1]. Providing a reliable estimation of structural responses in multiple intensities is one of the major obstacles in seismic damage cost assessment of structures. Various simplified procedures for seismic analyses have been used by researchers in order to overcome the huge computational demand involved in assessment of several design alternatives. Nevertheless, cost assessment has been mostly used in comparative studies among a limited number of design alternatives and, recently, direct incorporation of life cycle cost in design process has attracted the attention of researchers [1,28,29]. Push-over analysis has been widely used as seismic assessment tool in this area. However, well known limitations of this analytical tool besides its weaknesses in estimating floor accelerations to quantify non-structural cost components have increased the need for more realistic and reliable dynamic analysis procedures with tolerable computational demand. In this section, a procedure to calculate the expected cost components using ET analysis results has been formulated. Application of this method in optimum design of structures in the framework of next-generation performance based seismic design considering inherent uncertainties is studied by Basim and Estekanchi [30].

The total cost C_{TOT} of a structure can be considered as the sum of its initial construction cost, C_{IN}, which is function of design vector, s, and the present value of the life cycle cost, C_{LC}, which is function of lifetime, t, and the design vector, s [1]:

$$C_{\mathrm{TOT}}(t,s) = C_{\mathrm{IN}}(s) + C_{\mathrm{LC}}(t,s). \quad (1)$$

6.1. Initial costs

Initial cost is the construction cost of a new structure or the rehabilitation cost of an existing facility. In our design example, which is a new moment resisting steel frame, the initial cost is related to the land price, material, and the labor cost for the construction of the building. As the land price and non-structural components cost are constant for all design alternatives, they can be eliminated from the total cost calculation and the initial steel weight of the structure with a labor overhead can be considered as representor of the initial cost. Thus, an initial cost equal to \$500 per m^2 over the 700 m^2 total area of the structure for the prescriptive design is considered and for other design alternatives, it will be calculated according to their steel weight difference by a material plus labor cost of 2 \$/kg.

6.2. Life cycle cost

Life cycle cost in this study refers to the costs resulting from earthquakes that may occur during lifetime of the structure. Based on the recent literature, multiple limit states according to inter-story drift ratio are considered. These limit states and damages depend on the performance of both structural and nonstructural components. In order to calculate the life cycle cost of the structure, the following cost components are involved: the damage repair cost, the cost of loss of contents due to structural damage quantified by the maximum inter-story drift and also floor acceleration, the loss of rental cost, the loss of income cost, the cost of injuries, and the cost of human fatalities [18,31]. Some other factors have been defined and used by researchers to characterize earthquake consequences. Some of these factors try to quantify the impacts in the whole community level such as losses due to morbidity or loss in quality of life for the population [19] or resiliency measures [32]. In this study, since the main objective is to introduce the value based design methodology and explore the advantages of the ET method in this context, a simple cost model is used and readers are encouraged to refer to the provided references for more detailed models of earthquake consequences.

A correlation is required to quantify these losses in economic terms. Several damage indices have been used to quantify seismic performance of structures. Commonly, inter-story drift (Δ) has been considered as a measure of both structural and non-structural

Table 2. Drift ratio and floor acceleration limits for damage states.

Performance level	Damage states	Drift ratio limit (%) ATC-13 [33]	Floor acceleration limit (g) [34]
I	None	$\Delta \leq 0.2$	$a_{\text{floor}} \leq 0.05$
II	Slight	$0.2 < \Delta \leq 0.5$	$0.05 < a_{\text{floor}} \leq 0.10$
III	Light	$0.5 < \Delta \leq 0.7$	$0.10 < a_{\text{floor}} \leq 0.20$
IV	Moderate	$0.7 < \Delta \leq 1.5$	$0.20 < a_{\text{floor}} \leq 0.80$
V	Heavy	$1.5 < \Delta \leq 2.5$	$0.80 < a_{\text{floor}} \leq 0.98$
VI	Major	$2.5 < \Delta \leq 5$	$0.98 < a_{\text{floor}} \leq 1.25$
VII	Destroyed	$5.0 < \Delta$	$1.25 < a_{\text{floor}}$

damage. In this study, seven limit states according to drift ratios based on ATC-13 [33] are used to describe structural performance as shown in Table 2. On the other hand, maximum floor acceleration is used to quantify the loss of contents. The relation between floor acceleration values and damage states is shown in Table 2 based on a work by Elenas and Meskouris [34]. The addition of the maximum floor acceleration component in life cycle cost calculation is introduced by Mitropoulou et al. [18]. Piecewise linear relation has been assumed in order to establish a continuous relation between damage indices and costs [7].

Expected annual cost has been found to be the most proper intermediate parameter to calculate life cycle cost of structures using ET method. The procedure and formulation to calculate the expected cost components in ET framework are described here in details based on a common framework whose validity is investigated by Kiureghian [35]. The framework for performance based earthquake engineering, used by researchers at the Pacific Earthquake Engineering Research (PEER) Center, can be summarized by Eq. (2), named as PEER framework formula. By use of this equation, the mean annual rate (or annual frequency) of events (e.g., a performance measure) exceeding a specified threshold can be estimated by [35]:

$$\lambda(dv) = \int_{dm}\int_{edp}\int_{im} G(dv|dm)|dG(dm|edp)|| dG(edp|im)||d\lambda(im)|, \quad (2)$$

where:

- im — An intensity measure (e.g., the peak ground acceleration or spectral intensity);
- edp — An engineering demand parameter (e.g. an inter-story drift);
- dm — A damage measure (e.g. the accumulated plastic rotation at a joint);
- dv — A decision variable (e.g., Dollar loss, duration of downtime).

Here, $G(x|y) = P(x < X|Y = y)$ is the Conditional Complementary Cumulative Distribution Function (CCDF) of random variable X, given $Y = y$, and $\lambda(x)$ is the mean rate of $\{x < X\}$ events per year. The deterioration of the structure has been ignored here and it has been assumed that it is instantaneously restored to its original state after each damaging earthquake.

A fundamental assumption made is that, conditioned on EDP, DM is independent of IM, and, conditioned on DM, DV is independent of EDP and IM. Thus, it would be possible to decompose the earthquake engineering task into subtasks presented in Figure 9. The ET method is used in response analysis box in this flowchart and it will create a proper baseline to calculate the following boxes.

By considering various cost components as the decision variable, dv, in Eq. (2), $\lambda(dv)$, i.e. the annual rate that the cost component values DV exceed a value dv, can be obtained. Results can be presented by a curve with cost values of dv on the horizontal axis and

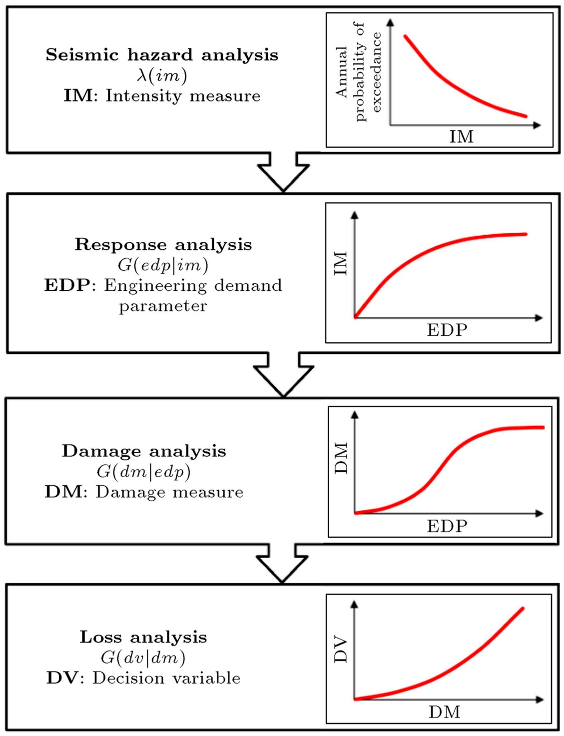

Figure 9. Performance based earthquake engineering framework [36].

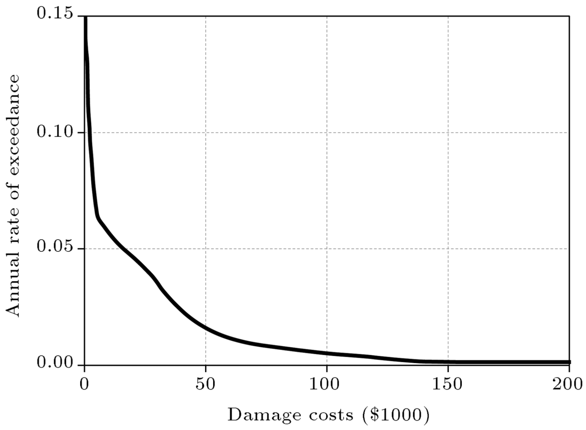

Figure 10. A sample loss curve due to damage cost.

annual rate of exceedance on the vertical axis, known as "Loss Curve" [36].

For variable X, the differential quantity $|\lambda(x+dx)-\lambda(x)| \cong |d\lambda(x)|$ describes the mean number of events $\{x < X \leq x+dx\}$ per year. Thus, assuming that X is non-negative, its expected cumulative value in one year is:

$$E[\sum X] = \int_0^\infty x|d\lambda(x)| = \int_0^\infty \lambda(x)dx. \quad (3)$$

Therefore, the area underneath $\lambda(x)$ versus x curve gives the mean cumulative value of X for all earthquake events occurring in one year. In our problem, where x is the cost component values as the decision variable, the area under $\lambda(dv)$ versus dv curve (i.e., Loss Curve) represents the mean cumulative annual component cost for all earthquake events in one year.

Loss Curve can be obtained from ET curve presented above in a practical procedure. First, the annual probability of exceedance of drift ratios should be determined. By reversing the return period on the x-axis to obtain the mean annual rate of exceedance and using it on the y-axis, the annual rate of exceedance of the inter-story drift can be obtained. If the inter-story drift is replaced by component cost applying the linear relationship discussed previously using Table 2, the annual rate of exceedance for the cost component, namely Loss Curve, can be obtained. The procedure to calculate Loss Curve for losses caused by floor acceleration is similar. In Figure 10, a sample loss curve due to damage cost is depicted. The area under the loss curve represents the mean annual component cost caused by all earthquakes in one year.

As mentioned, life cycle cost consists of several components and can be calculated as follows:

$$C_{\mathrm{LC}} = C_{dam} + C_{con} + C_{ren} + C_{inc} + _{inj} + C_{fat}, \quad (4)$$

$$C_{con} = C_{con}^{\Delta} + C_{con}^{acc}, \quad (5)$$

where $5C_{dam}$ is the damage repair cost; C_{con}^{Δ} the loss of contents cost due to structural damage quantified by inter-story drift; C_{can}^{acc} the loss of contents cost due to floor acceleration; C_{ren} the loss of rental cost; C_{inc} the cost of income loss; C_{inj} the cost of injuries; and C_{fat}

Table 3. Formulae for calulation of the cost components in Dollars [1,31,33].

Cost component	Formula	Basic cost
Damage repair (C_{dam})	Replacement cost × floor area × mean damage index	400 $/m^2
Loss of contents (C_{con})	Unit contents cost × floor area × mean damage index	150 $/m^2
Loss of rental (C_{ren})	Rental rate × gross leasable area × loss of function time	10 $/month/m^2
Loss of income (C_{inc})	Income rate × gross leasable area × down time	300 $/year/m^2
Minor injury ($C_{inj,m}$)	Minor injury cost per person × floor area × occupancy rate × expected minor injury rate	2000 $/person
Serious injury ($C_{inj,s}$)	Serious injury cost per person × floor area × occupancy rate × expected serious injury rate	20000 $/person
Human fatality (C_{fat})	Human fatality cost per person × floor area × occupancy rate × expected death rate	300000 $/person

Table 4. Damage state parameters for cost calculations [33,37].

Damage states	Mean damage index (%)	Expected minor injury rate	Expected serious injury rate	Expected death rate	Loss of function time (days)	Down time (days)
(I)-None	0	0	0	0	0	0
(II)-Slight	0.5	0.00003	0.000004	0.000001	1.1	1.1
(III)-Light	5	0.0003	0.00004	0.00001	16.5	16.5
(IV)-Moderate	20	0.003	0.0004	0.0001	111.8	111.8
(V)-Heavy	45	0.03	0.004	0.001	258.2	258.2
(VI)-Major	80	0.3	0.04	0.01	429.1	429.1
(VII)-Destroyed	100	0.4	0.4	0.2	612	612

the cost of human fatality. The formulae to calculate each cost component can be found in Table 3. The first term of each formula is presented in the last column of the table as the basic cost. The values of the mean damage index, loss of function, downtime, expected minor injury rate, expected serious injury rate, and expected death rate used in this study are based on ATC-13 [33] restated in FEMA-227 [37]. Table 4 provides these parameters for each damage state. Loss of function time and down time are considered as the time required to recover the full functionality of the building based on a table from ATC-13 [33] for earthquake engineering facility classification 16 and medium rise moment resisting steel frame. Also, Occupancy rate is taken 2 persons per 100 m^2. Note that these are an estimation of cost components and a detailed assessment is necessary to evaluate the expected cost. The method, with no limitation, has the capability of incorporating detailed calculation of cost components.

According to Eq. (1), the total life cycle cost is considered as the sum of the initial construction costs and the present value of the annual damage costs summed up through the lifetime of the structure. A discount rate equal to 3% over 50 years life of the building has been considered to transform the damage costs to the present value. This total cost is used as the objective function in optimization algorithm seeking a design with the least total cost.

As in the previous sections, Genetic Algorithm (GA) has been used to find the optimum design. Alternative designs should meet some initial constraints. Strong-column/weak-beam criterion should be checked and strength of columns should preserve a decreasing trend along the frame height. Besides these constraints, all AISC360 checks must be satisfied for the gravity loads. Once the expressed constraints are satisfied, the LCC analysis is performed. It is important to note that each of these feasible organisms is acceptable design according to the code ignoring seismic actions. In order to reach the optimum solution, the algorithm will reproduce new design alternatives based on the initial population and will mutate until the stop criterion is met. The flowchart of the applied methodology is presented in Figure 11.

Genetic algorithm with an initial population size of 200 leads to an optimum design after about 2600 ET

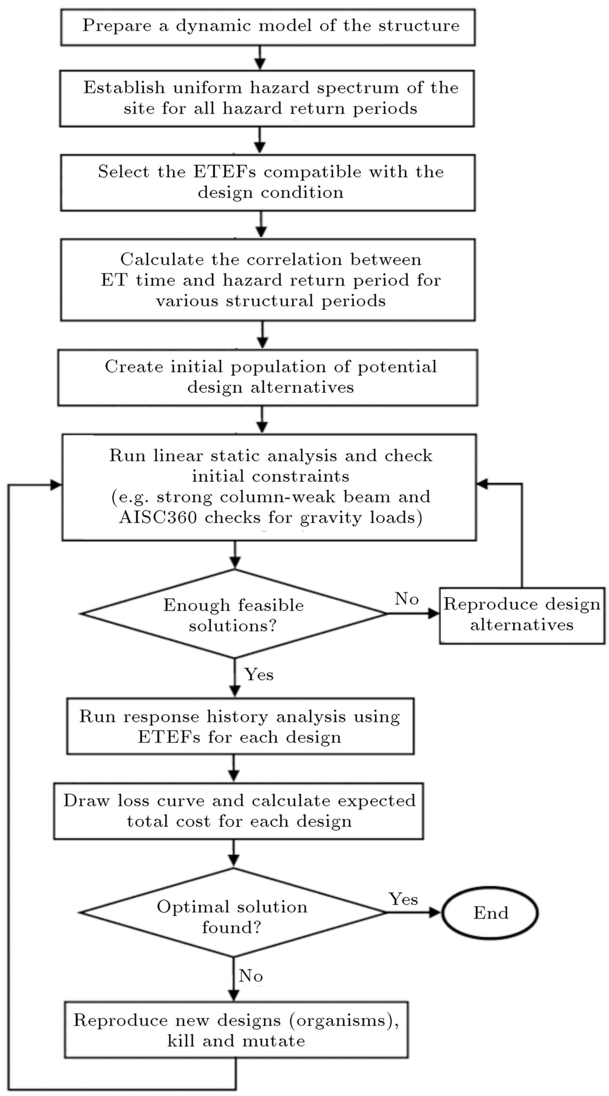

Figure 11. Flowchart of the value based design by the ET method.

response history analyses. Figure 12 shows the total costs for feasible design alternatives in optimization procedure. The optimum design sections are presented in Figure 13. The seismic performance of the optimum design according to FEMA-350 measures is investigated in Figure 14. According to this figure, this design satisfies performance limitations of FEMA-350 with a margin that can be justified by economic concerns.

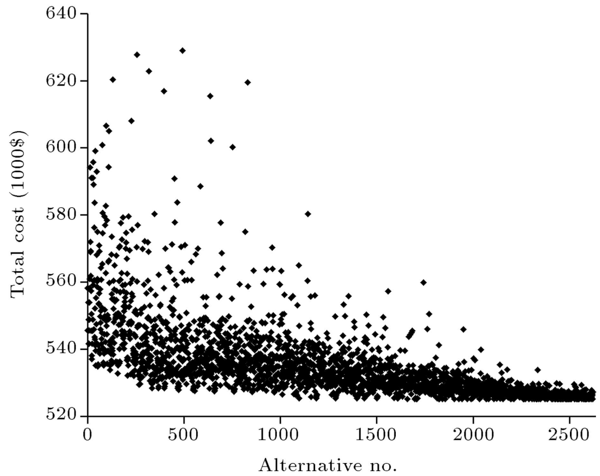

Figure 12. Total costs for feasible design alternatives in optimization procedure.

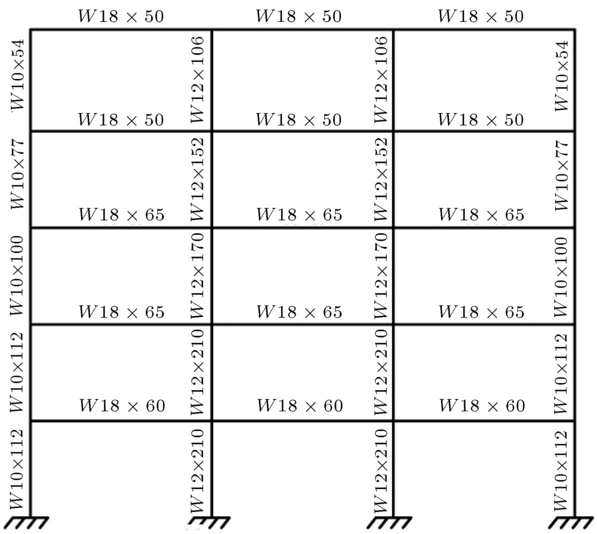

Figure 13. Value based design sections of the frame.

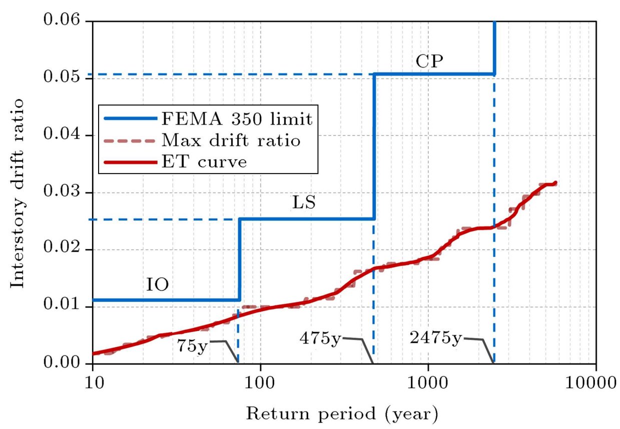

Figure 14. Performance curve (ET curve) for the value based design.

7. Comparative study

In this section, components of life cycle cost for the three structures (i.e. prescriptive, performance based, and value based designs) are compared. These structures are designed optimally based on various design philosophies. In Figure 15, cost components for the three structures are provided in 1000$. Each bar presents contribution of various cost components and the value of total cost for each design can be found above the bars. Components in bars are in the same order as that of the legend for the sake of clarity. As it can be seen, the prescriptive design has the least initial cost but the largest total cost among three designs and the value based design, having a larger initial cost, has the least total cost in long term. Also, the value based design has a larger cost of content loss due to floor acceleration. It may reaffirm the sophistications involved in selecting a desired design alternative. In Table 5, initial costs based on the used initial material, present value of life cycle costs due to seismic hazards with various exceedance probabilities, and the determinative part, i.e. total cost of three structures, are presented. It can be verified that a value based design has the least total cost and would be an economical alternative in long term. An extra initial cost of 12200$ over the prescriptive design will

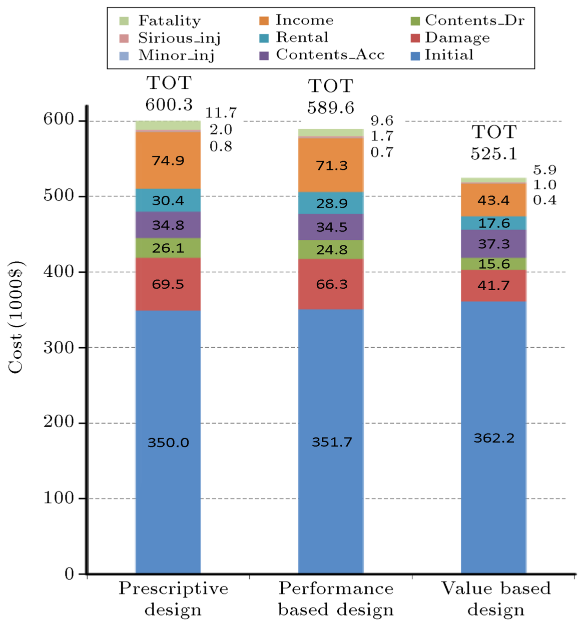

Figure 15. Cost components and total cost for the three designs (1000$).

Table 5. Values of life cycle cost terms for the three designs (1000$).

Design type	Initial cost	Life cycle cost	Total cost
Prescriptive	350	250.3	600.3
Performance based	351.7	237.9	589.6
Value based	362.2	162.9	525.1

lead to a decrease by 87400$ in the expected life cycle cost totally having 75200$ of profit. Although performance based design has less expected total cost than the prescriptive design, neither the prescriptive design criteria nor the performance based ones will necessarily lead to an economical design in long term.

8. Conclusions

A framework that directly acknowledges the concept of value in the structural design procedure is established using the advantages of Endurance Time (ET) method. Application of the ET analysis in Life Cycle Cost Analysis (LCCA) has been formulated in a general form which has the capability to be extended using more detailed cost models of the construction. ET method and the resultant performance curve have provided a proper baseline to calculate expected life cycle costs, while the required computational effort is in an acceptable range to be used in conventional optimization techniques. This will provide the means to extend the economic concerns from a merely appraisal tool to a more impressive role which directly defines the design targets. To demonstrate the method and compare it with other common design methods, a five-story moment frame has been optimally designed according to three distinct design philosophies: a prescriptive design code, a performance based design guideline, and the introduced methodology named Value Based Design of structures (VBD). A case-specific cost model has been defined to quantify the consequences of probable earthquakes. The procedure to calculate life cycle cost of the structure using ET results has been presented and used in an optimum design algorithm. Structural performance and life cycle cost components for the three optimum designs have been compared using ET curve. Results show that the code based design of the structure will not necessarily result in an economical design with less total cost in lifetime of the structure. For the studied building, performance based design requires more initial material cost than the prescriptive design because of its more restricting limitations and, as expected, it has better performance in various hazard intensities. However, the value based design has the least total cost among the three designs, although it demands the highest initial material cost. The proposed methodology provides a pathway towards practical value based seismic design. It also shows that conventional design procedures based on compliance to design code requirements or performance objectives do not assure achievement of the best final design regarding the overall applicable design concerns. However, this study aimed to introduce a methodology for optimum design based on financial considerations, and more research is required for defining appropriate models of earthquake consequences, especially models to account for uncertainties.

Nomenclature

a_{floor}	Floor acceleration
ATC	Applied technology council
CCDF	Conditional Complementary cumulative Distribution Function
CP	Collapse Prevention
C_{con}	Loss of contents cost
C_{con}^{acc}	Loss of contents cost due to floor acceleration
C_{con}^{Δ}	Loss of contents cost due to inter-story drift
C_{dam}	Damage repair cost
C_{fat}	Cost of human fatality
C_{inc}	Loss of income cost
C_{inj}	Cost of injuries
$C_{inj,m}$	Cost of minor injuries
$C_{inj,s}$	Cost of serious injuries
C_{ren}	Loss of rental cost
C_{IN}	Initial cost
C_{LC}	Life cycle cost
C_{TOT}	Total cost
dm	A damage measure threshold
dv	A decision variable threshold
DM	Damage Measure
DV	Decision Variable
edp	An engineering demand parameter threshold
E	Elastic modulus
EDP	Engineering Demand Parameter
ET	Endurance time
ETEF	Endurance Time Excitation Function
FEMA	Federal Emergency Management Agency
F_y	Yielding stress
g	Acceleration of gravity
GA	Genetic Algorithm
im	An intensity measure threshold
IDA	Incremental Dynamic Analysis

IM	Intensity Measure
IO	Immediate Occupancy
INBC	Iranian National Building Code
LCCA	Life Cycle Cost Analysis
LRFD	Load Resistance Factor Design
LS	Life Safety
PBD	Performance Based Design
PEER	Pacific Earthquake Engineering Research center
R	Behavior factor
s	Design vector
t	Lifetime of structure
t_{ET}	ET excitation time
VBD	Value Based Design
Δ	Inter-story drift ratio

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