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APPLICATION OF ENDURANCE TIME METHOD IN VALUE BASED SEISMIC DESIGN OF STRUCTURES

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ABSTRACT

In this study the concept of a new design methodology based on the total value of the structure namely Value Based Design of structures (VBD) by the use of potential benefits of the ET method has been introduced and formulated. Commonly, prescriptive and also performance based approaches of seismic design try to find a structure with lower initial cost satisfying a number of requirements under seismic actions. The objective of the VBD methodology is to incorporate directly economic concerns in design procedure to achieve an economical design with lower total cost in its life time. In this approach the design sections of structural elements are assumed as optimization parameters and the optimization goal is to minimize the total cost of the structure during its life span resulting in maximized value of the structure in its life time. Reduced computational demand in ET analysis method provides the prerequisites to use optimization algorithms in design procedure. The optimization has been conducted through genetic algorithm. For each possible candidate design the damages due to probable earthquakes in its life time is estimated by the ET method and the expected cost of damages is calculated using Life Cycle Cost Analysis (LCCA). A prototype 3 story steel frame has been designed according to a prescriptive design code, performance based design criteria and the proposed value based design method and then seismic performance and cost components of these structures are investigated and discussed.

INTRODUCTION

Large economic losses following earthquakes and hurricanes in recent decades have shown the need for improved design criteria and procedures that provide the necessities to reduce damage and economic impacts to an acceptable level along with life protection. The prescriptive and also performance based approaches of seismic design try to find a structure to satisfy minimal requirements under seismic actions in a number of levels of intensity and a design with lower initial cost is normally preferred. These approaches will not necessarily result in an economical design with lower total cost in life time of the structure. On the other hand, Life Cycle Cost Analysis (LCCA) has provided a reliable tool for estimating damage cost due to future earthquakes during the design life of a structure. Thus, to incorporate directly the economic concerns in design or decision making procedure LCCA has been applied in construction industry. This analysis in companion with an optimization algorithm can result in a design with the least total cost. Considering economic and also technical issues in design and construction field will lead to optimum allocation of the public resources. Although in construction industry LLCA was firstly introduced in economical investment assessment of infrastructures, nowadays, LCCA becomes an essential component of the design process used to control the initial and the future cost of buildings in seismically active regions and is widely used in risk assessment and decision analysis. By the use of this method the expected total cost of a structure including the initial

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cost and also losses resulting from earthquakes during its life span can be considered as the main representer of the priority of design alternatives. In this paper, LCCA is used to determine the total value of a structure as an investment appraisal tool to be incorporated in design procedure.

To calculate expected life cycle cost of a structure using LCCA the cost components related to the performance of the structure in multiple hazard levels should be estimated (Mitropoulou et al. 2011). In order to have a reasonably reliable performance assessment and estimate the seismic capacity of a structural system to be incorporated into the LCCA methodology, response-history based incremental analyses and considering a realistic numerical model of the structure are inevitable. However, these procedures require repetitive and massive analyses and their huge computational demand and sophistications involved may make optimization algorithms impractical or the simplifications used decrease the reliability of the outcome. In this paper, Endurance Time (ET) method, as a dynamic procedure requiring reasonably reduced computational effort, is applied to estimate the performance of the structure in various hazard levels (Estekanchi et al. 2004). In the ET method, structures are subjected to specially designed intensifying acceleration functions instead of a set of progressively scaled up ground motions in IDA and their performance is assessed based on their response at different excitation levels correlated to specific ground motion intensities by each single response-history analysis. Thus, the required huge computational demand of a complete response history analysis is considerably reduced while maintaining the major benefits of it, i.e. accuracy and insensitivity to model complexity (Estekanchi and Basim 2011). Application of ET method in combination with the concept of LCCA can pave the way for practical Value Based seismic Design of structures.

The endurance time (ET) method introduced by Estekanchi et al. (2004) as an analysis method, can be utilized to assess seismic performance of the structures in a continuous range of seismic hazard intensities. In this method, structures are subjected to a predesigned gradually intensifying accelerogram and the seismic performance of the structure can be monitored while the seismic demand is increasing. Application of the ET method in performance assessment of structures has been studied by Mirzaee et al. (2010). Reasonably accurate estimates of expected seismic response at various excitation intensities of interest have been obtained through ET analysis by correlating the dynamic characteristics of intensifying excitations with those of ground motions at various hazard levels (Mirzaee and Estekanchi 2013).

To demonstrate the method a three story and one bay steel special moment frame is optimally designed according to Iranian National Building Code (INBC), which is almost identical to the ANSI/AISC360 (2010) LRFD design recommendations. Also, the frame is designed optimally to conform to FEMA350 (2000) limitations as a performance based design criteria. The performance of the designed frames is investigated by the ET method and as a third step a new design sections has been acquired through the introduced method to have the minimum total cost during its life time that is assumed 50 years. The resultant prescriptive, performance based and value based designs of the frame are different due to their distinct basic design philosophies. Seismic performance and cost components of these structures are investigated and discussed.

BACKGROUND

Most of the current seismic design codes belong to the category of the prescriptive design codes which consider a number of limit state checks to provide safety. The two accepted limit state checks are serviceability and ultimate strength. Commonly, a design with a lower weight or initial material cost is commonly preferred. Prescriptive building codes do not provide acceptable levels of a building life-cycle performance, since they only include provisions aiming at ensuring adequate strength of structural members and overall structural strength (Mitropoulou et al. 2011). To fulfill the deficiencies of the primitive design procedures, design codes are migrating from prescriptive procedures intended to preserve life safety to reliability-based design and most of them have attempted to advance their design criteria towards Performance Based Design (PBD) of structures. The report of the SEAOC committee in 1995 can be entitled as the start of this progress. Performance based earthquake engineering states the methodology in which structural design criteria are expressed in terms of achieving a set of different performance objectives defined for different levels of excitations where they can be related to the level of structural damage. In this methodology the performance of the building after construction is inspected in order to ensure reliable and predictable seismic performance over its life. In PBD more accurate and time-consuming analysis procedures are employed in order to estimate the entire non-linear structural response. Various guidelines on performance based design concept have been introduced over the last 10 years for assessment and rehabilitation of existing structures and the analysis and design of new ones. FEMA350 (2000) supplies a probability based guideline for performance based design of new steel moment resisting frames considering uncertainties in seismic hazard and structural analyses. Design codes based on reliability of performance are useful in providing safety margins for the performance objectives with quantifiable confidence levels considering various sources of uncertainties.

The first step in performance-based design is selecting the performance objectives and developing a preliminary design. Then, seismic response of the design is evaluated. Afterwards, the design can be revised until the acceptance criteria for all intended performance objectives are satisfied. In order to achieve optimal structural designs with acceptable performance, optimization methods have been effectively used for PBD while the structural performances and also structural weight incorporated as objective functions or constraints to the optimization problem (Ganzerli et al. 2000). Among many others, Pan et al. (2007) incorporated multiple design requirements into a multi-objective programming problem using a new formulation based on the constraint approach and Liu et al. (2013) utilized a performance based design approach for a multi-objective optimization using genetic algorithm subjected to uncertainties and provided a set of Pareto-optimal designs.

Prescriptive and performance based design of structures lack the capability to incorporate the economic issues in design process and commonly in engineering practice a design alternative with lower initial cost is normally preferred. Large economic losses following recent earthquakes and hurricanes encouraged researchers to introduce financial concerns in structural design area. The LCCA principles are based on economic theories and it was mainly implemented to energy and water conservation projects as well as transportation projects. However, LCCA has become an important part of structural engineering to assess the structural comeback and evaluate the performance of the structure during its life span in economic terms and has gained considerable attention of decision making centers to decide on the most cost effective solution related to the construction of building structures in seismic regions. Firstly, LCCA was applied in the commercial area and in particular in the design of products. Later in early 2000s, as one of the impressive works in this area, Wen and Kang (2001) formulated long term benefit versus cost considerations for evaluation of the expected life cycle cost of an engineering system under multiple hazards. Many works has been accomplished later to take the advantages of economic accounts in structural engineering. Liu et al. (2003) defined a multi-objective optimization problem and an automated design procedure to find optimal design alternatives with respect to three objectives. Static pushover analyses were performed to verify the performance of steel frame design alternatives and genetic algorithm was used. Takahashi et al. (2004) formulated the expected life-cycle cost of design alternatives using a renewal model for the occurrence of earthquakes in a seismic source, which accounts for the temporal dependence between the occurrence of characteristic earthquakes and applied the methodology to an actual office building as a decision problem. Liu et al. (2005) formulated performance-based seismic design of steel frame structures as a multi-objective optimization problem considering the seismic risk in terms of maximum interstory drift. Fragiadakis et al. (2006) used pushover analysis to compare single objective optimal design of minimizing the initial weight and a performance based two objective designs of a steel moment resisting frame; in particular a framework to generate a Pareto front of the solutions were presented. Kappos and Dimitrakopoulos (2008) used cost benefit and life-cycle cost analyses as decision making tools to examine the feasibility of strengthening reinforced concrete buildings. Mitropoulou et al. (2010) probed the influence of the behavior factor in the final design of reinforced concrete buildings under earthquake loading in terms of safety and economy by demonstrating initial and damage cost components for each design. Mitropoulou et al. (2011) investigated the effect of the analysis procedure, the number of seismic records imposed, the performance criterion used and the structural type on the life-cycle cost analysis of 3D reinforced concrete structures. Furthermore, the influence of uncertainties on the seismic response of structural systems and their impact on LCCA is examined using Latin hypercube sampling method.

Endurance Time method (ET)

In the Endurance Time (ET) method, structures are subjected to a predesigned intensifying dynamic excitation and their performance is monitored continuously as the level of excitation is increased. ET excitation functions are in the form of artificial accelerograms created in such a way that each time window of them from zero to a particular time produces a response spectrum that matches a template spectrum with an increasing scale factor. This interesting characteristic has been achieved by resorting to numerical optimization procedures in producing ET accelerograms (Nozari and Estekanchi 2011). Various sets of ET acceleration functions have been produced with different template response spectra and are available through the ET website (Estekanchi 2014). 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 can be seen in Figure 1, the response spectrum of a window from t=0 to t=10 sec of used accelerogram matches with the template spectrum. Furthermore, the produced response spectra also match the template spectrum at all other times with a scale factor, thus producing a correlation between analysis time and induced spectral intensity. Hence, each ET analysis time is representative of a particular seismic intensity and results can be more effectively presented by considering a correlation between time in ET analysis and the equivalent hazard return period based on code recommendations considering the fact that hazard levels are well presented by acceleration response spectra in current codes. This can result in an appropriate baseline to calculate probabilistic damage and cost.

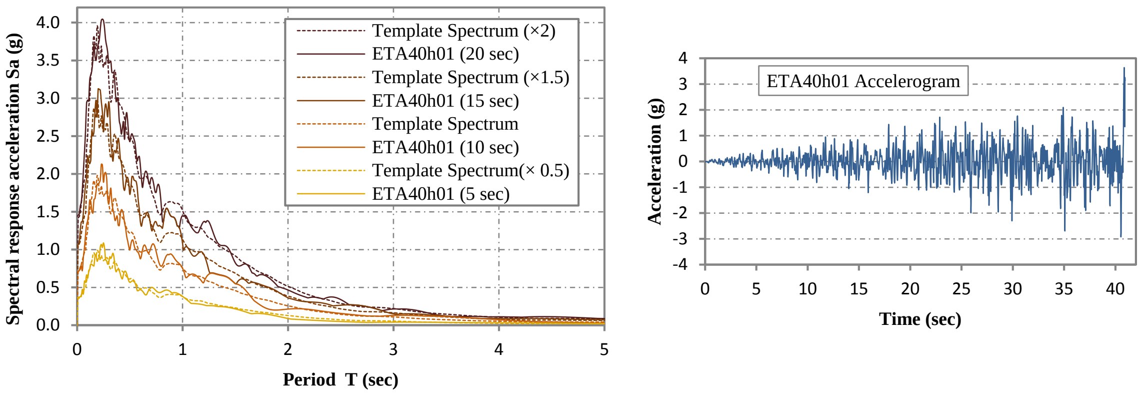

Figure 1. Accelerogram and acceleration response spectra for ETA40h01 at different times.

The application of the ET method in performance-based design was studied by Mirzaee et al. (2010) introducing “Performance curve” and the “Target Curve”, which expresses respectively the seismic performance of a structure along various seismic intensities and their limiting values according to code recommendations. Substituting return period or annual probability of exceedance for time in the expression of the performance will make the presentation of the results more explicit and their convenience for calculate probabilistic cost will be increased.

Hazard return period corresponding to a particular time in ET analysis can be calculated by matching the response spectra at effective periods, e.g. from 0.2 to 1.5 times of structure’s fundamental period. The procedure is based on the coincidence of response spectra obtained from the ET accelerogram at different times and response spectra defined for Tehran at different hazard levels. The results show that substitution of the return period for time in ET analysis and performance curves increases the usefulness of these curves and can simplify application of the ET method in value-based design. Figure 2 illustrates the variation of the return period with the structural period and time in ET analysis. Using this correlation for a specific structure the corresponding ET time for each hazard level is on hand. The detailed procedure to obtain such a correlation is explained in a work by Mirzaee et al. (2012). In Figure 3 a sample target and performance curve for the 3 story structure is depicted where ET analysis time has been mapped into return period on horizontal axis. As it can be seen the structure satisfies the code CP level limitations but it has violated the IO and LS levels limitations and the frame does not have acceptable performance. Also, moving average is applied to smooth ET results for interstory drift envelope curve. It can be inferred one of advantages of ET method that the performance of the structure in continuous increasing hazard levels can be properly depicted in an easy to read figure.

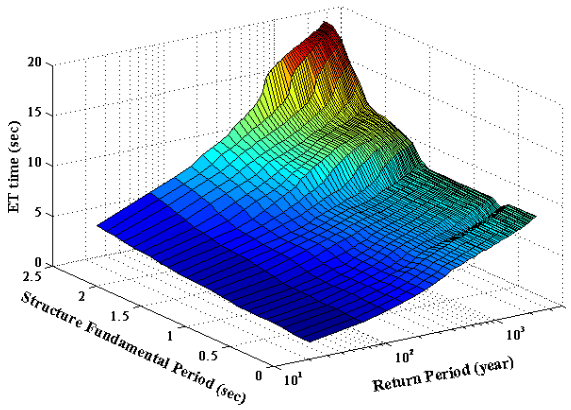

Figure 2. Return period vs. structural period and ET analysis time.

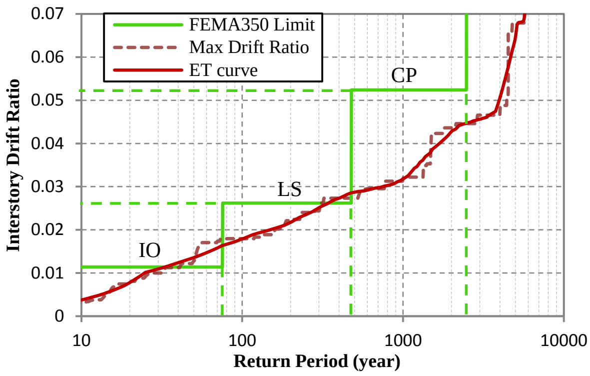

Figure 3. A sample performance curve (ET curve) for the three-story steel frame.

PRESCRIPTIVE SEISMIC DESIGN

In prescriptive seismic design procedures the structure is considered safe if it satisfies a number of checks in one or two deterministically expressed limit states (i.e. ultimate strength and serviceability). Also, the structures are allowed to absorb energy through inelastic deformation by designing them with reduced loading specified by the behavior factor leading to smaller seismic loads.

The under study structure has been designed according to the Iranian National Building Code (INBC), which is almost identical to the ANSI/AISC 360-10 LRFD design recommendations as a prescriptive design code. The prototype structure is a 3 story and one bay special moment resisting steel frame. The geometry of this model can be found in Figure 4. All supports are fixed and the joints are all rigid. The beams and columns are selected among seismically compact standard W profiles. Loading is set according to Iranian National Building Code section 6. The steel material considered has the property as yielding stress of F_y = 235.36 MPa, elastic modulus of 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 years return period hazard level and the elastic base shear is reduced by a behavior factor (R) to incorporate the inelastic deformation capacity of the special moment frame. Code limitations for interstory drift ratios have dominated the design in first and second stories. In this section, it has been tried to accomplish the design procedure in the same way as the procedure in common engineering practice. Demand/Capacity ratios are depicted in Figure 4. As can be seen in this figure in some elements other limitations such as drift limits or strong column-weak beam limitation is dominant.

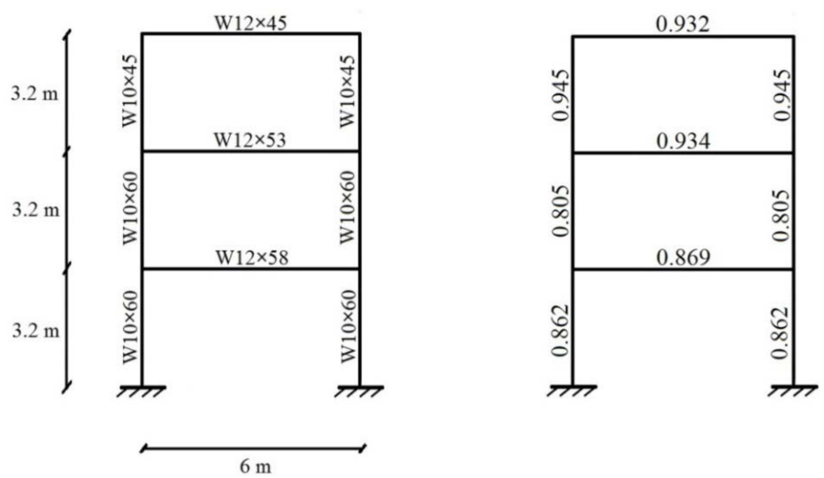

Figure 4. Schematics of steel frames under investigation and demand/capacity ratios according to prescriptive design.

The seismic performance of the prescriptive design has been investigated according to FEMA350 limitations on interstory drift ratios. The procedure and recommended limitations are explained in next section. Performance curve and also Target curve for this structure is depicted in Figure 5. It can be verified that the structure has violated IO level limitation but has a proper performance in LS and CP levels.

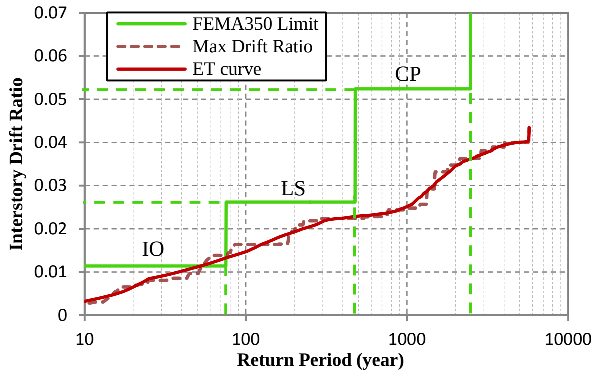

Figure 5. Performance curve (ET curve) for the prescriptive design.

PERFORMANCE-BASED DESIGN

Compared to prescriptive design procedures, Performance-Based Design (PBD) has provided a more general structural design philosophy in which the design criteria are expressed in terms of achieving multiple performance requirements when the structure is subjected to various seismic hazard levels. It is known that prescriptive design procedures do not provide reliable performance of the structure in multiple hazard levels during its life span, since these procedures intend to keep the ultimate strength of structural members at an acceptable level. In most of current performance based design criteria the structural performance of an ordinary building frame is usually defined as: (1) resist a significant accidental earthquake without structural damage, (2) allow repairable structural damage against a rare major earthquake, and (3) resist the maximum credible earthquake without collapse (Pan et al. 2007). The performance measures may include the response stresses, the maximum load carrying capacity, the interstory drift or the plastic rotation at members, etc. In the most common approaches to PBD, the performances against seismic motions are defined based on the displacements or global deformation. Various methods of structural assessment have been used by researchers and also engineers. Especially push-over analysis is widely being used in this area; however, time history analysis is so far believed to be the most accurate methodology for evaluating structural performance. In order to utilize performance based measures in design procedure and achieve a safe yet economic design, using optimization methods is inevitable. Merits of ET method in optimum performance based design and its methodology is introduced in a work by Estekanchi and Basim (2011).

The PBD procedure implemented in this work is based on the method introduced in FEMA350 (2000). This criteria supplies a probability based guideline for performance based design of new steel moment resisting frames, in which the ground motion variability and the uncertainty in the structural analysis are considered explicitly. FEMA 350 considers two discrete structural performance levels, Collapse Prevention (CP) and Immediate Occupancy (IO) by introducing the limiting damage states for common framing elements related to these performance levels and acceptance criteria are related to the permissible interstory drifts and earthquake-induced forces for the various elements especially in columns. Interstory drift ratio is a commonly used measure of both structural and non-structural damage because of its close relationship to plastic rotation demands on individual beam-column connection assemblies. As recommended in these criteria 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 have been used by interpolating the IO and CP levels. LS level is a damage state in which significant damage has been sustained, although some margin remains against either partial or total collapse. The considered performance objective in this study assuming seismic use group I for the prototype special moment frame structure is IO, LS and CP performance levels corresponding to ground motion levels of 50%, 10% and 2% probability of being exceeded in 50 years, respectively.

Many uncertain factors exist that affect the behavior and response of a building such as uncertainties in seismic hazard due to the attenuation laws employed, record to record variability or on the other hand uncertainties in structural modeling due to simplifications and assumptions used in the numerical analysis (Liu et al. 2013). Therefore, FEMA 350 adopts a reliability-based probabilistic approach to performance evaluation that explicitly acknowledges 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 70% and 90% for IO and CP performance levels, respectively, the upper bound limit for calculated interstory drift demand obtained from structural analysis would be 0.0114 and 0.0524 and interpolation will result in an upper bound of 0.0262 for LS level.

Structural analysis has been performed by OpenSees (Mazzoni et al. 2006) where the nonlinear behavior is represented using the concentrated plasticity concept with zerolength rotational springs and structural elements are modeled using elastic beam-column elements. The rotational behavior of the plastic regions follows a bilinear hysteretic response based on the Modified Ibarra Krawinkler Deterioration Model (Ibarra et al. 2005, Lignos and Krawinkler 2010). Second order effects have been considered using P-Delta Coordinate Transformation object embedded in the platform. To capture panel zone shear deformations, panel zones are modeled using the approach of Gupta and Krawinkler (1999) as a rectangle composed of eight very stiff elastic beam-column elements with one zerolength rotational spring in the corner to represent shear distortions in the panel zone.

In this section a single objective optimization problem is defined to find a design having the minimum initial steel material weight as optimization objective and according to FEMA350 recommendations, as performance based design criteria, the limitations on interstory drift demand and axial compressive load on columns and also strong column weak beam criterion 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 code such as specifications of AISC360 (2010) and AISC Seismic (AISC341 2010). Thus, the AISC 360 requirements and FEMA 350 acceptance criteria are implemented as 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 (2011). The acquired design sections can be found in Figure 6. Performance of the structure in various seismic intensities can be investigated using ET curve presented in Figure 7. Eventually, an optimum design will meet the constraints (i.e. code requirements) with the least margins.

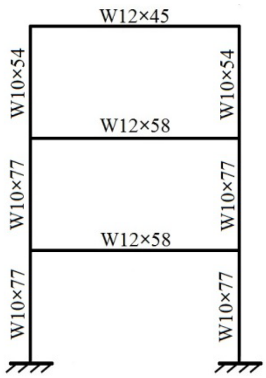

Figure 6. Performance based design sections of the frame.

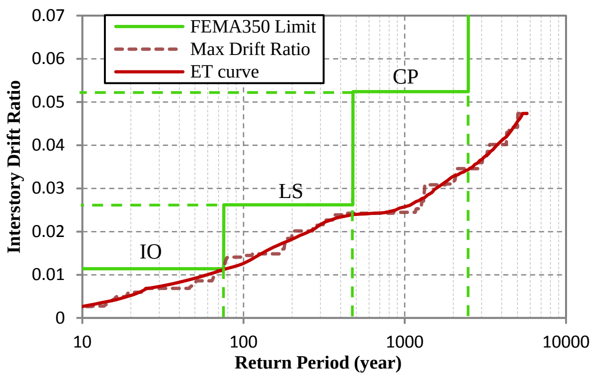

Figure 7. Performance curve (ET curve) for the performance based design.

VALUE-BASED DESIGN

For the clarity of explanation, in this research we consider the structure that is more economical to construct and maintain, to be the most valued. But, value can be defied and considered in its broad sense for design purposes. ET analysis provides a versatile baseline to perform economical analyses on design alternatives with acceptable computational cost. Initial construction cost and expected seismic damage cost throughout the life-time of the structure are usually the two most important parameters for decision making (Mitropoulou et al. 2011). One of the major obstacles in seismic damage cost assessment of structures is response estimation of structures subject to ground motions in multiple intensities. Various simplified procedures for seismic analyses have been used by researchers in order to overcome huge computational demand involved in assessment of several design alternatives. Nevertheless, cost assessment has been mostly used in comparative study among limited number of design alternatives and incorporation of life cycle cost directly in design process has attracted the attention of researchers (Frangopol et al. 2009, Mitropoulou et al. 2011, Kaveh et al. 2012). Push-over analysis has been widely used as seismic assessment tool in this area. However, well known limitations of this analytical tool besides its disability in estimating non-structural cost components due to floor acceleration have increased the need for more realistic and reliable dynamic analysis procedures with a tolerable computational demand. In this section, ET analysis has been used to estimate seismic response of the structure and the procedure to calculate the required cost components has been formulated.

In this study, since the aim is to demonstrate the concept of the method, the costs corresponding to damage states is obtained as a percentage of the initial cost from a table introduced in ATC-13 (1985) (Table 1). This is a very rough estimate of cost components and a detailed assessment is necessary to evaluate the expected cost, especially, human fatality and injury related losses has a determinative portion in expected costs. The method, with no limitation, has the capability of incorporating detailed calculation on cost components and will be investigated in further studies. Piecewise linear relation has been assumed in order to establish a continuous relation between interstory drift and cost (Mirzaee and Estekanchi 2013).

Table 1. Damage states, drift ratio limits and corresponding costs (ATC-13 1985).

Performance level	Damage states	Drift ratio limit (%)	Cost (% of initial cost)
I	None	$\Delta < 0.2$	0
II	Slight	$0.2 < \Delta < 0.5$	0.5
III	Light	$0.5 < \Delta < 0.7$	5
IV	Moderate	$0.7 < \Delta < 1.5$	20
V	Heavy	$1.5 < \Delta < 2.5$	45
VI	Major	$2.5 < \Delta < 5$	80
VII	Destroyed	$5.0 < \Delta$	100

Expected annual damage cost found to be the most proper intermediate parameter to calculate life cycle cost of structures using ET method. The procedure and formulation whose validity is investigated by Kiureghian (2005) to be used in ET framework is described here in detail.

A common framework for performance-based earthquake engineering, used by researchers at the Pacific Earthquake Engineering Research (PEER) Center, can be summarized by formula (1) named as PEER framework formula. By the use of this formula the mean annual rate (or annual frequency) of events (e.g. a performance measure) exceeding a specified threshold can be estimated (Kiureghian 2005).

$$\lambda(dv) = \int_{dm}\int_{edp}\int_{im} G(dv|dm)\left|dG(dm|edp)\right|\left|dG(\mathrm{edp}|im)\right|\left|d\lambda(im)\right| \tag{1}$$

Where:

im : an intensity measure (e.g. the peak ground acceleration)
edp : an engineering demand parameter (e.g. an interstory drift)
dm : a damage measure (e.g. the accumulated plastic rotation at a joint)
dv : denotes a decision variable (e.g. dollar loss, duration of downtime)

Here $G(x|y) = P(x < X \mid Y = y)$ denotes 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. All of the aleatory and epistemic uncertainties present in describing the model of the structure and its environment and also stochastic nature of earthquakes can be properly modeled in this formula. It should be noted that the deterioration of the structure has been ignored and it has been assumed that it is instantaneously restored to its original state after each damaging earthquake. Another fundamental assumption made is that, conditioned on EDP, DM is independent of IM, and, conditioned on DM, DV is independent of EDP and IM. The later assumption makes it possible to decompose the earthquake engineering task into subtasks presented in Figure 8. Note that the ET method is used in response analysis box in this flowchart and will create a proper baseline to calculate the following boxes.

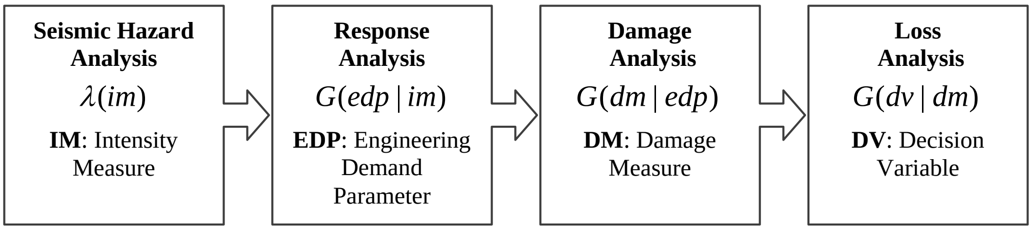

Figure 8 Performance-based earthquake engineering framework (Yang et al. 2009).

Herein, by considering repair cost values as decision variable dv and using formula (1), $\lambda(dv)$ the annual rate that the repair cost values DV exceeds a value dv can be obtained. Results can be presented by a curve with repair cost values dv in horizontal axis and annual rate of exceedance as vertical axis known as “Loss Curve” (Yang et al. 2009).

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

$$E[\sum X] = \int_0^\infty x |d\lambda(x)| = \int_0^\infty \lambda(x)\, dx \tag{2}$$

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

As a practical procedure, Loss Curve can be acquired from ET curve mentioned above. Here, 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 interstory drift can be obtained. If the interstory drift is replaced by damage cost applying the linear relationship discussed previously using Table 1, and considering the initial cost equal to \$300 per m^2 over the 200 m^2 total area of the structure, the annual rate of exceedance for damage cost can be obtained as shown in Figure 9. This curve is the Loss Curve being sought. The area under the loss curve represents the mean annual total damage cost caused by all earthquakes in one year.

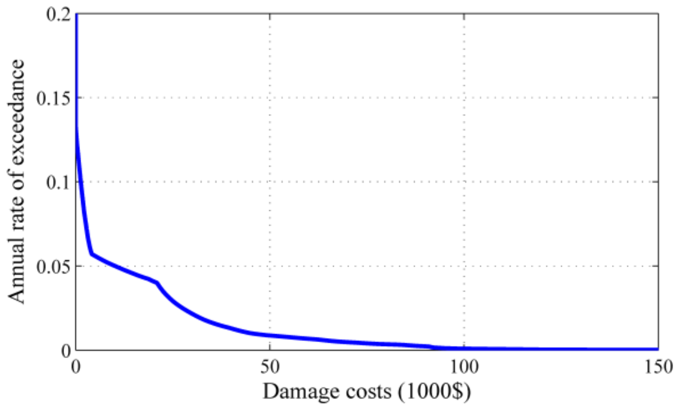

Figure 9. A sample Loss Curve for the 3 story frame.

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 life time of the structure. A discount rate equal to 3% over a fifty-year life of the building has been considered to transform the damage costs to the present value and calculate the expected damage cost of the structure in its life time. This total cost is used as the cost function in optimization algorithm and due to capabilities of genetic algorithm this design is the global optimum alternative with a high chance.

The total cost of the structure is selected as the optimization objective to be minimized. As the previous sections, genetic algorithm (GA) has been used to find the optimum design. Alternative designs should meet some initial constraints. One of the constraints is strong column and weak beam criterion which should be checked and the other constraint that should be considered before the analysis phase is that the selected sections for columns in each story should not be weaker than the upper story. Beside these constraints, all AISC 360 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. But, in order to reach the optimum solution, algorithm will reproduce new design alternatives based on the initial population and mutate until the stop criteria is met.

Genetic algorithm with an initial population size of 100 leads to an optimum design after about 1800 ET response history analyses. Total costs for feasible design alternatives in optimization procedure are depicted in Figure 10. The optimum design and its performance in various seismic intensities (i.e. ET curve) is presented in Figure 11 and Figure 12. It can be seen that the structure satisfies performance limitations of FEMA350 with a margin that is justified by economic concerns.

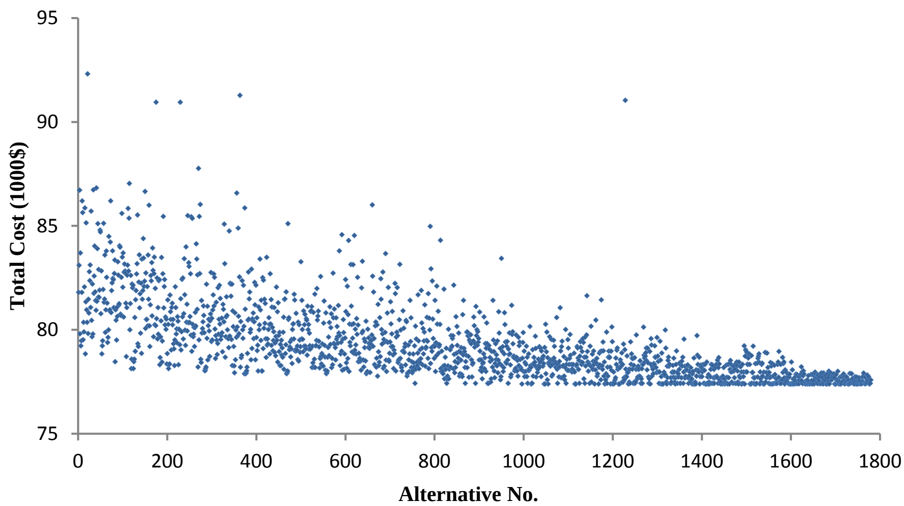

Figure 10. Total costs for feasible design alternatives in optimization procedure.

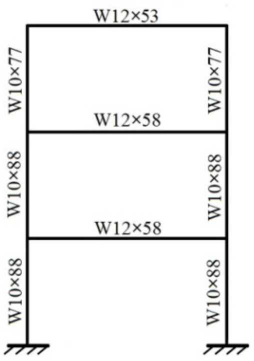

Figure 11. Value based design sections of the frame.

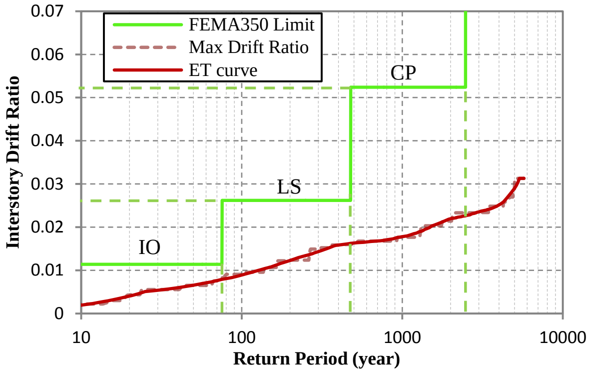

Figure 12. Performance curve (ET curve) for the value based design.

COMPARATIVE STUDY

Here, initial costs and life cycle costs for the three structures (i.e. prescriptive, performance based and value based designs) are compared. These structures are design optimally based on various design philosophies. In Table 2 initial costs based on used initial material, present value of damage 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 increase of 2300$ in initial material cost over the prescriptive design will lead to a decrease of 4600$ in expected damage cost having totally 2290$ profit. Although Performance based design has a minor expected total cost in comparison with the prescriptive design, neither the prescriptive design criteria nor the performance based one will necessarily lead to an economical design in long term.

Table 2. Values of life cycle cost terms for the three designs (1000$).

Design Type	Initial cost	Damage cost	Total Cost
Prescriptive	60.14	19.55	79.69
Performance based	61.22	17.77	78.99
Value based	62.44	14.95	77.40

CONCLUSIONS

A methodology to incorporate the concept of value in the structural design procedure using the benefits of Endurance Time (ET) method has been established. Application of the ET analysis in Life Cycle Cost Analysis (LCCA) has been formulated. ET method and resultant performance curve has provided a proper baseline to calculate expected damage cost, while the required computational effort is in an acceptable range to be used in conventional optimization techniques. To demonstrate the method and compare it with prescriptive and performance based design criteria, a three story moment frame has been optimally designed according to three distinct design philosophies: a prescriptive design code, a performance based design guideline and also the introduced methodology namely Value Based Design of structures (VBD). Structural performance and life cycle cost components for the three structures have been compared. The resultant prescriptive, performance based and value based designs of the frame are different due to their distinct basic design philosophies. Results show that the code based design of the structure will not necessarily result in an economical design with lower total cost in life time of the structure. Performance based design in this case turns out to require higher initial material cost in comparison with the prescriptive design due to its more restricting limitations, and as expected, better performance in various hazard intensities. The value based design, however, demands the highest initial material cost, yet the least total cost among three, justifying the increased initial cost.

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