Optimal multi-objective design of a highway bridge under seismic loading through Incremental Dynamic Analysis

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ABSTRACT: A methodology is introduced for the optimal multi-criteria performance-based design of highway bridges under seismic loading based on the concept of Incremental Dynamic Analysis (IDA). IDA is a novel analysis method that can thoroughly estimate the seismic demands and limit-state capacities of a structure under seismic loads by subjecting it to a suite of ground motion records that are suitably scaled to several levels of intensity. The mean annual frequencies (MAFs) of exceeding each limit-state become readily available when combining the results with probabilistic seismic hazard analysis. By thus analyzing each alternate design we are able to directly apply the desirable constraints on the acceptable performance of the bridge and employ evolutionary strategies to perform Pareto optimization and minimize the bridge cost and the MAF of collapse. As an example, the Pareto set is generated for a typical two-span bridge with a single-column pier and a prestressed concrete deck supported on elastomeric bearings. It is shown that improving the bearings provides an inexpensive increase in collapse performance, but further gains necessitate costly strengthening of the pier column. The procedure presented is resource-intensive but highly accurate. It provides important information on the influence of design parameters on the bridge performance and allows their optimal selection.

1 INTRODUCTION

Recent advances in structural engineering have brought the emergence of performance-based design and the appearance of relevant guidelines both for buildings (e.g, FEMA-273, FEMA-350) and bridges (AREMA 2002, ATC/MCEER 2001, Caltrans 2002). Furthermore, large interest seems to be focused upon the use of optimization, specifically genetic algorithms and evolutionary strategies, to solve the difficult problem of structural design under seismic loads, especially when conflicting objectives are present, such as initial construction cost versus safety or versus life-cycle cost (e.g. see Frangopol et al 1985, Beck et al 1999, Papadrakakis et al 2002, Liu et al 2003).

In practically every case the use of multi-criteria genetic algorithms or evolutionary optimization techniques (Beyer 2001) has proven to be extremely robust and well-suited to solve design problems compared to the conventional gradient-based algorithms. However, the slow convergence rate of such methods and the requirement for many functional evaluations before the optimum is reached has led to the use of simpler and often less accurate methods to analyze the alternate designs under seismic loads. For example, in Papadrakakis et al (2002) an elastic response spectrum modal analysis is used, while Liu et al (2003) prefer a static pushover methodology using a bilinear single-degree-of-freedom representation of the structure.

While such methodologies offer unmatched computational advantages, especially for preliminary design, they can be highly inaccurate away from the early inelastic region, especially close to dynamic instability where the structure is about to collapse. On the other hand the recent emergence of powerful analysis techniques represents another possible tradeoff. For example Incremental Dynamic Analysis (IDA, Vamvatsikos & Cornell 2002), offers superb accuracy at the cost of several nonlinear dynamic analyses under multiply-scaled ground motions. Our aim is to use IDA on a typical highway bridge and investigate the possibility of matching such a resource intensive analysis procedure with probabilistic optimization techniques for optimal design in a performance-based framework.

2 PROBLEM SETUP

The design problem is formulated in a multi-objective context that allows the simultaneous minimization of the multiple objectives, eliminating the need for using arbitrary weighting factors to weigh the relative importance of each objective. For conflicting objectives there is no single optimal solution, but rather a set of alternative solutions which are optimal in the sense that no other solutions in the search space are superior to them when all objectives are considered. Such alternative solutions, in our case trading-off the cost and safety of the bridge, are known in multi-objective optimization as Pareto optimal solutions. The set of Pareto solutions can be obtained using Evolutionary Algorithms (Beyer 2001) well-suited to solve multi-objective optimization problems (Fonseca & Fleming 1995, Srinivas & Deb 1994). Casting the problem in a format suitable for multi-criteria optimization means defining the objectives and the constraints to be used. Our choice will be to (a) define meaningful performance levels, or limit-states, for the bridge and (b) define constraints and objectives based directly on the limit-states and the cost.

Regarding the bridge performance levels, there does not seem to be a consensus yet on what limits to use for performance-based design (Yashinsky & Karshenas 2003). Probably the most relevant guidelines come from the NCHRP Project 12-49 (ATC/MCEER 2001) where two levels of performance are defined for an ordinary bridge, namely an "immediate service level" for an earthquake with 50% in 75 years occurrence, where the bridge is required to sustain minimum damage, and a "significant disruption level" for a 3% in 75 years earthquake, where the bridge may be usable after shoring but could very well be replaced later. Similar two-level criteria are also suggested by the Caltrans Seismic Design Criteria (2002), while three-level criteria are suggested for railway bridges (AREMA 2002).

The adopted approach in defining the limit-states stems from the draft ideas presented by the Pacific Earthquake Engineering Research Center in co-operation with Caltrans (Porter 2002). Therein it is suggested to use four performance levels: Immediately Operational (IO) where no action needs to be taken, Operational (O) where the bridge may have to be closed for a few days and some repairs may be needed, Life Safe (LS) when lateral capacity has been impaired, the bridge has to be closed for an extended time and serious repair work and shoring is needed and finally Collapse Prevention (CP), where the bridge needs to be closed and may later be repaired or replaced, whichever is cheaper. These correspond to distinct limit-states but they are difficult to define using structural response variables. As will be discussed in a later section, we have defined the limit-states using reasonable limits for the maximum pier column drift and the maximum bearing displacement and shear strain. These choices are not restrictive and only represent an engineering decision that may be revised easily.

Furthermore, taking advantage of the collapse prediction capabilities inherent in IDA (Vamvatsikos & Cornell 2002) we have defined the Global Instability (GI) limit-state. It occurs when the deck falls off the abutment seat or pier seat, or when the pier has reached dynamic instability due to excessive loading, whichever occurs first. This event obviously has a higher (or at most equal) return period than the CP limit-state and when it occurs the bridge has collapsed and needs to be replaced.

When defining the constraints and objectives for the design, it makes sense to choose from these five limit-states plus some measure of the bridge cost. The main idea is that the bridge must satisfy some basic acceptability criteria for the constrained limit-states but the designer will seek possible optimal tradeoffs regarding the unconstrained performance levels versus the cost. Thus, different selections of what to constrain and what to optimize may correspond to very different design schemes. If we are willing to spend money for a bridge that will not close as often, but has in general some fixed MAF of facing serious repairs, means constraining O, LS, CP, GI and optimizing for cost and IO. If on the other hand we are satisfied with some standard limits for frequent small-scale repairs but wish to trade money for the collapse safety of the bridge, then we need to constrain IO, O, LS, CP and optimize for cost and GI.

The latter case was the choice for the example; therefore, we will constrain the IO, O, LS and CP limit-states by directly setting occurrence rates that are deemed reasonable for this ordinary bridge. Specifically, the IO level was set at 75% in 50 years maximum, the O level at 50% in 50 years, the LS level at 10% in 50 years and the CP level at 2% in 50 years. The corresponding minimum allowed return periods are 36, 72, 475 and 2475 years respectively. On the other hand, we will let the optimization pursue the optimal tradeoff for the GI limit-state and the material construction cost. The cost is defined as the combined material cost of the bearings, the concrete and the steel, using typical prices adopted by designers in Greece.

Formally, if we let C be the material cost, λ_{L} be the mean annual frequency of exceeding limit-state "L", $\tau_{\mathrm{L}} = 1/\lambda_{\mathrm{L}}$ the associated return period and $\boldsymbol{\theta}$ the parameter vector, we have posed the following optimization problem:

$$\begin{aligned}
&\text{minimize } C(\boldsymbol{\theta}), \lambda_{\mathrm{GI}}(\boldsymbol{\theta}),\\
&\text{where } \boldsymbol{\theta} \in \Theta \qquad\qquad (1)\\
&\text{subject to } \begin{cases} \tau_{\mathrm{IO}} > 36\,\text{yrs} \\ \tau_{\mathrm{O}} > 72\,\text{yrs} \\ \tau_{\mathrm{LS}} > 475\,\text{yrs} \\ \tau_{\mathrm{CP}} > 2475\,\text{yrs} \end{cases}
\end{aligned}$$

where Θ is the parameter space.

The optimization may further be constrained due to restrictions imposed to the elements of the parameter vector $\boldsymbol{\theta}$. The feasible parameter space Θ is usually confined in a hypercube by specifying lower and upper limits on each parameter. These limits depend on physical constraints, information about the physical characteristics of the system and modeling experience, and they are going to be discussed in conjunction with the bridge description in the following section.

3 HIGHWAY BRIDGE MODEL

As the testbed for our methodology we will employ a typical highway bridge, shown in Fig. 1, to be constructed in a high seismicity region in Greece. It is a reinforced concrete structure that has a total length of 68m and width of 13.30m. It caries two lanes of traffic on a reinforced concrete superstructure made of four precast, pretensioned beams connected by a cast-in-place concrete deck slab (Fig. 2). There is one pier made of a single hollow rectangular reinforced concrete column (Fig. 3) that is 28m tall. The superstructure is made up of two identical spans supported on elastomeric bearings both on the pier cap and on the seat-type abutments. There is one bearing under each of the four beams on either side of each span and there are also 30cm wide stoppers in the longitudinal direction to control displacement. They allow a maximum displacement of 0.5m before the deck hits them, and a total of 0.8m of displacement, if broken, before the deck slides off the pier cap or the abutment seat.

The parameters of the problem that have been selected for optimization define the stiffness and strength of the bearings and of the pier column. For the (square) bearings these are the width b and elastomer height h, both for the abutment bearings (b_a and h_a) and the pier bearings (b_p, h_p). For the pier column we have included the longitudinal width (along the bridge axis) of the hollow rectangular pier section, b_c, and the longitudinal (height-wise) reinforcement ratio ρ_c. Thus, the parameter vector for the optimization problem defined in Eq. 1 is:

$$\boldsymbol{\theta} = \left(b_a, h_a, b_p, h_p, \rho_c, b_c\right) \tag{2}$$

These six parameters are allowed to vary freely within some reasonable engineering limits. Specifically, we set the minimum limits for the bearing parameters according to some minimum designer requirements while the maxima were set by taking into account the catalogue of sizes typically manufactured. Thus we allowed the width of the bearings b_a and b_p to vary within 0.65m and 1.0m and the total rubber height h_a and h_p to have values within 8cm and 25cm. For the column width we set a minimum according to code requirements and expert opinion, so we let it vary between 2m and 4m. Finally, the code supplied limits were used for the reinforcement, i.e., the allowable range is 1%-4%.

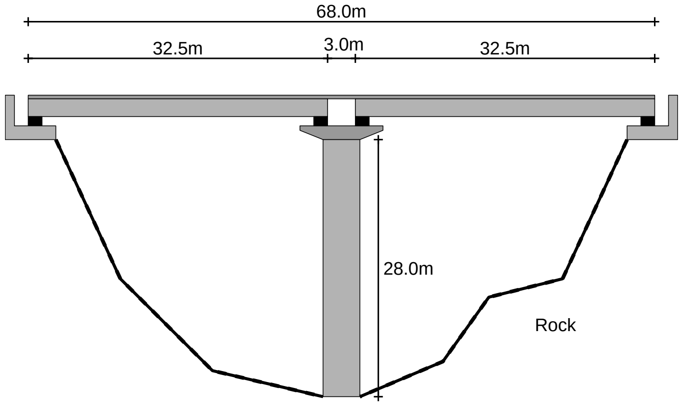

Figure 1. The bridge to be designed

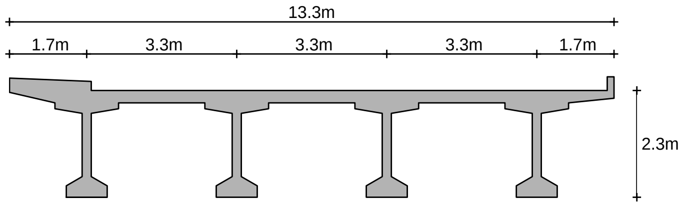

Figure 2. Section of the bridge deck, comprised of four precast, pretensioned I-beams and a cast-in-place deck slab

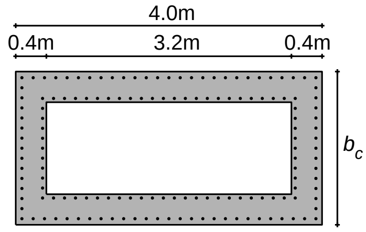

Figure 3. The hollow rectangular section for the pier column: 4m wide in the transverse direction, b_c width in the longitudinal direction and an all around concrete thickness of 0.40m.

4 PERFORMANCE EVALUATION WITH IDA

Incremental Dynamic Analysis (IDA) is an emerging analysis method that offers thorough seismic demand and capacity prediction capability (Vamvatsikos & Cornell, 2002). It involves performing a series of nonlinear dynamic analyses under a multiply scaled suite of ground motion records, selecting proper Engineering Demand Parameters (EDPs) to characterize the structural response and an Intensity Measure (IM), e.g. the 5% damped first-mode spectral acceleration, $S_a(T_1, 5\%)$, to represent the seismic intensity, and then generating curves of EDP versus IM for each record. On such IDA curves the appropriate limit-states can be defined by setting appropriate limits on the EDPs and the probabilistic distribution of their capacities can be estimated. Such results combined with probabilistic seismic hazard analysis (Vamvatsikos & Cornell 2002) allow the estimation of mean annual frequencies (MAFs) of exceeding the limit-states. These are exactly what is needed to characterize performance and measure the ability of any candidate design to withstand rare or more frequent seismic threats. IDA has been applied both to structures (Vamvatsikos & Cornell 2002) and bridges (Mackie & Stojadinovic 2002) and it has been shown to provide excellent prediction capabilities, forming a reliable tool for assessing each alternate design.

Here we are going to explain in detail the elements needed to run IDA, i.e., the structural model and the record suite. Then we will explain how the limit-states are defined and finally we will present the application of IDA for the bridge model for a given selection of the design parameters. In essence, we will take the reader briefly through all the steps needed to perform the performance evaluation of a single design case.

4.1 *Structural model*

We need an accurate structural model and a robust analysis program that can track the bridge's performance in all the ranges of structural response, from elasticity to collapse. Our choice was the OpenSEES software (McKenna et al 2000) which allows extremely reliable and realistic modeling. We have thus chosen complex models for the elastomeric bearing that incorporate the degradation and eventual breaking of the rubber at high strains, the existing gap between bearings and stoppers and the finite strength of the latter. A detailed fiber model was used for the reinforced concrete section of the pier column while an accurate representation of geometric nonlinearities (P-Δ effects) was included using an exact corrotational formulation.

4.2 *Record suite*

In order to simulate the seismic threat we are going to use ten records, shown in Table 1, to perform Incremental Dynamic Analysis on each candidate design. These were selected from a relatively narrow magnitude and distance bin, having moment magnitude within 6.5 – 6.7 and closest distance to fault rupture 18 – 38km. They have all been recorded on firm soil and bear no marks of near-fault directivity. In essence they represent the typical scenario threat associated with the high seismicity area that the bridge is to be designed for.

While objections may be raised due to the limited size of the record suite, it should be noted that this is a first-mode dominated, medium period structure that we are dealing with, the period ranging within 0.9 – 1.3s. So, at least we are far from the short-period range where the record-to-record variability can be quite large and ten records would not be enough. Of course, more records will always mean greater accuracy and a higher confidence in the results. Still, increasing the number of records would proportionately increase the computation time. This can easily make the optimization problem impossible to complete within a reasonable time limit. However, if a higher accuracy is desired, it could be easily accommodated by performing a more detailed analysis with more records (twenty or thirty) on all design alternatives that are close to the Pareto front, thus defining it more accurately.

Table 1. The suite of records used for IDA

Event Station	φ[1] deg	Soil[2]	M[3]	R[4] km	PGA g
Superstition Hills, 1987					
Plaster City	135	C,D	6.7	21.0	0.19
Brawley	225	C,D	6.7	18.2	0.16
San Fernando, 1971					
LA Hollywood Sto Lot	180	C,D	6.6	21.2	0.17
Imperial Valley, 1979					
Chihuahua	012	C,D	6.5	28.7	0.27
Plaster City	135	C,D	6.5	31.7	0.06
Compuertas	285	C,D	6.5	32.6	0.15
Northridge, 1994					
Leona Valley #2	090	C,-	6.7	37.7	0.06
Lake Hughes #1	000	C,C	6.7	36.3	0.09
LA Hollywood Sto FF	360	C,D	6.7	25.5	0.36
LA Baldwin Hills	090	B,B	6.7	31.3	0.24

[1] Component
[2] USGS, Geomatrix soil class
[3] Moment magnitude
[4] Closest distance to fault rupture

4.3 *Defining the limit-states*

There are two elements of the bridge that can sustain damage from earthquakes: The bearings and the pier column. Therefore, any limit-state definition needs to take into account limiting values for both of them. In specific for the bearing we need to keep track of the maximum absolute shear strain, $\gamma_{\max}$, and the maximum absolute displacement normalized by the bearing width, $\delta_{\max}$. According to code guidelines, the rubber may fracture at high values of $\gamma_{\max}$ necessitating the replacement of bearings, while $\delta_{\max}$ provides us with a rule-of-thumb to determine the lateral resistance degradation of the bearing under a given axial load. Additionally we have to take into account the "positive" bearing displacement $d^{+}_{\max}$, defined as the displacement away from the bearing seat. This measures exactly the separation between the deck and the abutment or the pier; if it exceeds 0.8m it will result in the deck falling off. Regarding the column we only need to track the peak drift $\theta_{\max}$, which has been shown to correlated well with column damage and is frequently used in this role (e.g., Mackie & Stojadinovic 2002). In total we have four different EDPs, all of which can play a part in deciding the violation of each limit-state.

Specifically, IO is set to occur when $\theta_{\max} = 1\%$ (initiation of cover spalling) or $\delta_{\max} = 0.33$ or $\gamma_{\max} = 120\%$ (mild bearing damage), whichever occurs first. O is similarly defined at $\theta_{\max} = 2\%$ (large visible cracks), $\delta_{\max} = 0.5$ or $\gamma_{\max} = 180\%$ (serious bearing damage), LS occurs when $\theta_{\max} = 3\%$ (degraded column capacity) or $\delta_{\max} = 0.75$ and finally CP appears at $\theta_{\max} = 5\%$ (serious degradation of column capacity) or $\delta_{\max} = 1.0$ (bearing has moved beyond its footprint). The GI limit-state will appear only if one of the two failure modes happens: Either $d^+_{\max} = 0.8\text{m}$, i.e., the deck falls off the pier or the abutment seat due to excessive displacement and breaking of the stopper (strong column, weak bearings), or $\theta_{\max} = +\infty$, i.e., dynamic instability appears in the columns (weak column, strong bearings). In the vast majority of cases though, only the first event happens, the second appearing only in few, if any, of the possible designs.

4.4 *Performing the analysis*

Performing IDA for each record involves several dynamic nonlinear timehistory analyses under suitable scaling, selected to cover the entire range of bridge behavior, from elasticity to final collapse. After each analysis the four EDPs have to be recovered and plotted to generate a single IDA curve for each EDP and each record.

As an example we will show the results for a candidate design having $b_c = 3.0\text{m}$ pier width, $\rho_c = 2.0\%$ reinforcement ratio, $b_a = 0.7\,\text{m}$ and $h_a = 0.12\,\text{m}$ for the abutment bearings and $b_p = 0.8\,\text{m}$, $h_p = 0.17\,\text{m}$ for the pier bearings. This case has a first mode period $T_1 = 1.2\text{s}$ and a material cost $C = 106{,}573$. By performing seven analyses per record, recording the EDP values and interpolating with a flexible spline scheme (Vamvatsikos & Cornell 2004) we get ten IDA curves for each EDP. Fig. 4 shows the IDA curves for $d^+_{\max}$. When these reach 0.8m, a flatline occurs, as the deck has fallen off and the bridge is now considered to have collapsed. These flatlines set the maximum IM limit that the design can withstand and they are the same (in IM terms) for all EDPs. In Fig. 5 are the curves for $\delta_{\max}$ which actually look quite similar to the curves of $d^+_{\max}$, which means that the absolute and the "positive" value are well correlated in this case. Finally in Fig. 6 we can see the results for the maximum drift $\theta_{\max}$. In all cases there is considerable dispersion in the results while the column and bearing EDPs seem well correlated, something to be expected for this rather simple system.

By applying the EDP-based definitions of the limit-states on the IDA curves we can easily estimate the limit-state capacities in IM-terms for each record. Then, using the Pacific Earthquake Engineering Research Center framing equation (Cornell & Krawinkler 2000) plus an appropriate hazard curve for $S_a(T_1, 5\%)$ (Fig. 7) we can integrate to get MAFs of exceeding each limit-state (Vamvatsikos & Cornell 2004).

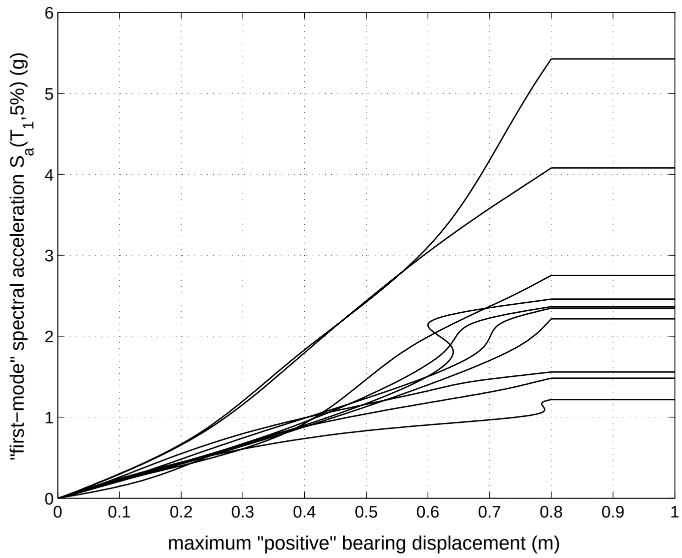

Figure 4. Example IDA curves for the "positive" bearing displacement $d^+_{\max}$, i.e. the maximum separation of the deck and the pier or abutment.

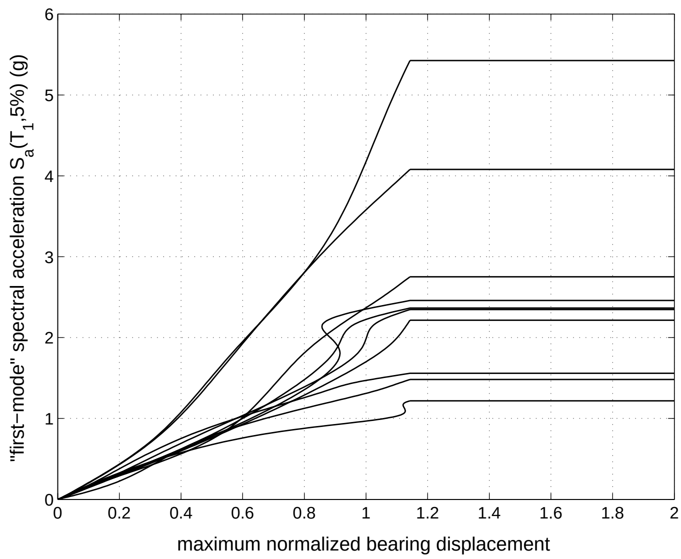

Figure 5. Example IDA curves for maximum bearing displacement normalized by the bearing width, $\delta_{\max}$.

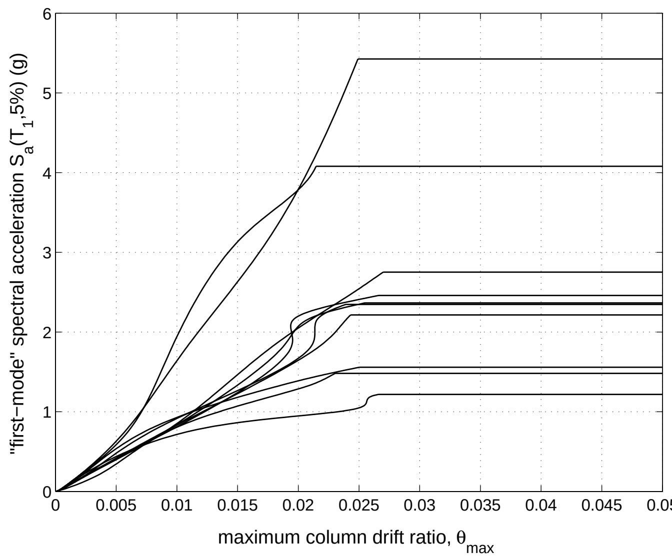

Figure 6. Example IDA curves for a sample design for column drift $\theta_{\max}$.

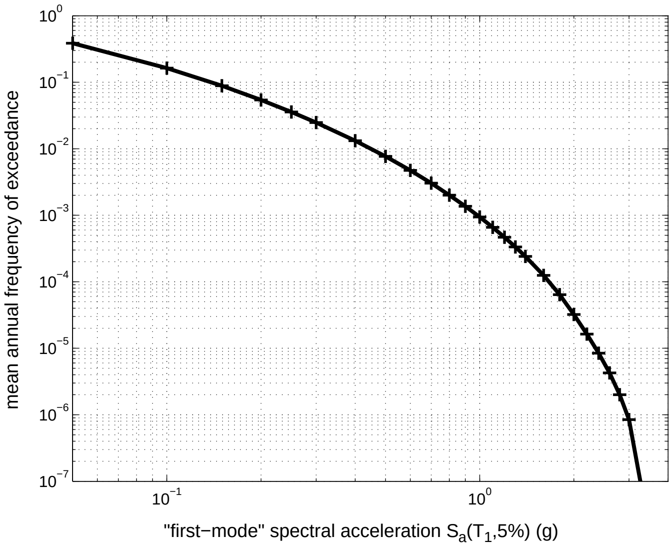

Figure 7. Typical hazard curve for a high seismicity area (Los Angeles, Van Nuys) for T=1.2s, corresponding to the bridge example case.

While the PEER framework is general enough to allow inclusion of epistemic uncertainties, we will neglect all material and model associated uncertainties, only retaining the uncertainty related to (and incorporated into) the hazard curve (see Cornell et al 2002). Having said that, if we perform the necessary numerical integration we come up with the following return periods $\tau_{IO} = 32$, $\tau_{O} = 93$, $\tau_{LS} = 951$, $\tau_{CP} = 1690$ and $\tau_{GI} = 3580$. Obviously the IO and CP constraints are violated, hence this case will be rejected by the algorithm. The total computing time on an older Pentium-III processor was only 11 minutes.

5 OPTIMIZATION

Evolutionary algorithms are well-suited to perform the multi-objective optimization. They process a set of promising solutions simultaneously and therefore are capable of generating multiple points along the Pareto front. These algorithms are based on an arbitrarily initialized population of search points in the parameter space, which by means of selection, mutation, and recombination evolves towards better and better regions in the search space. In this work, a recently proposed Strength Pareto Evolutionary Algorithm (SPEA) (Zitzler & Thiele 1999) based on evolution strategies is used for solving the multi-objective minimization problem.

The SPEA algorithm uses a number of features specific to multi-objective optimization algorithms (Fonseca & Fleming 1995, Srinivas & Deb 1994) for finding the multiple Pareto optimal solutions in parallel. Specifically, it stores in an external set the nondominated solutions (i.e., the best candidates for the Pareto set) found in each generation. It uses the Pareto dominance concept in order to assign fitness values to individuals. The fitness of an individual is determined only from the solutions stored in the external nondominated set. The solutions in the external set participate in the selection. It accomplishes fitness assignment and selection that guides the search towards the Pareto optimal set. It maintains diversity in the population so that a well-distributed, wide spread trade-off front is reached, preventing premature convergence to a part of the Pareto front (Mueller et al 2001). Finally, it performs clustering to reduce the number of nondominated solutions (Morse 1980).

In our bridge example, the SPEA algorithm was allowed to use a population size $\lambda = 60$, with $\mu = 9$ parents but retain only 18 points in the Pareto front and perform a total of 40 generations. The constraints (Eq. 1) were imposed in the strict sense, eliminating any offspring that would not satisfy them. Since this is a relatively small problem (only two objectives and six parameters) the algorithm easily converged to an acceptable Pareto set, but it still took about 18 days on an older Pentium-III class processor. Especially when closing to the optimal set the generations take longer to complete simply because the algorithm is searching closer to the constraints (were the minimal cost lies for a given collapse performance). This in turn causes a higher percentage of offsprings to be rejected and recreated in each generation. Hopefully, taking advantage of such information, instead of removing it, would help to improve convergence speed.

6 RESULTS

The Pareto front obtained appears in Fig. 8. Allowing the algorithm to perform more generations would probably smooth the front's shape quite a bit. Still, since we are aiming to do engineering design, rather than an abstract optimization exercise, it makes little sense to look for the bearing or column size parameters in the millimeter range; there is no need for such accuracy in practice.

Looking at the parameters of the Pareto front (Table 2), it becomes apparent that in all cases it is best for the bearings to be stiffer on the abutments than the pier. Also, as we move to higher costs and better collapse performance we invariably move to stiffer bearings (initially), which is a simple and cost-effective method for improving performance, but then, as we reach the limits of bearing sizes and the stiff bearings start transferring large forces, we also have to go to stiffer and better reinforced pier columns. This is a much more expensive option, and while it manages to improve performance it tends to increase the cost very quickly and had better be avoided. Still, if we examine the test cases that are not close to the Pareto front, we realize that this logic cannot be carried too far. If we allow the bridge pier to become too stiff, then the structure moves to shorter periods that fall into the more damaging areas of the records used. Thus there is a definite limit on the optimal collapse performance that has been imposed by the records and the nature of the structural system, and no matter how much we choose to pay we cannot exceed it.

A side result of these observations is that there are a lot of quite expensive designs that fare very poorly, sometimes even violating the constraints. The reason is that the pier column is what mainly drives the cost, while choosing inadequate bearings will always result in violation of the constraints. Thus, it is quite easy for the optimization algorithm to choose some thick and well reinforced column with too small bearings that will of course be rejected subsequently. Actually, this is one of the reasons that prompted us to set some rather strict minimum bearing sizes (at least 65cm by 65cm footprint area), so that we automatically escape most such cases. This has speeded up the process enormously.

We also see that improving the collapse performance does not always mean improving the performance in other limit-states as well. Actually when we stiffen the bearings all limit-states benefit. On the other hand, when the bearings are exhausted and the column sizes start increasing then the "safety" limit-states (LS, CP, GI) get a big boost but the "operational" ones (IO, O) usually worsen. For example, when $b_c = 3.25$m and $\rho_c = 2.7\%$ coupled with stiff bearings (one of the Pareto set points) then $\tau_{\mathrm{IO}} = 40$yrs, while $\tau_{\mathrm{GI}} = 125000$yrs. Another (non-Pareto set) point with relatively similar bearings but $b_c = 2.4$m and $\rho_c = 1.0\%$ shows $\tau_{\mathrm{IO}} = 65$yrs and $\tau_{\mathrm{GI}} = 9900$yrs. Effectively the operational and the safety limit-states are partially decoupled for this bridge; the parameters could be used to tune them separately.

Finally, as a word of caution, it is important to remember that we have not included material and model uncertainties. Had we done so, the design would come out more conservative (i.e., larger sizes in general).

7 CONCLUSIONS

The multi-criteria optimization procedure is shown to be a powerful method for performance-based seismic design. Using IDA as the tool for performance evaluation lends more accuracy and credibility to the analysis sub-process. It allows the direct application of constraints on the performance of a structure and provides a method for defining them that is well suited to current guidelines. By applying this procedure on a simple bridge case-study various interesting aspects of the structural behavior are uncovered, showing the influence of structural parameters such as the bearing and the pier properties on the bridge performance at several limit-states.

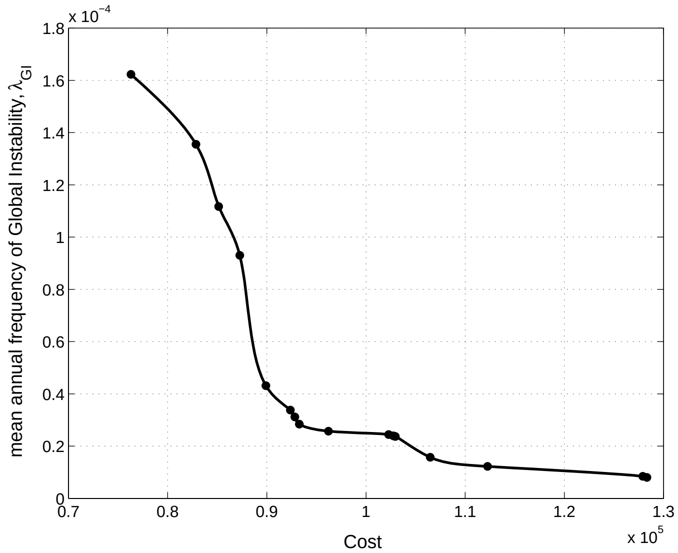

Figure 8. The Pareto front in the objectives' space.

Table 2. The Pareto set points in the parameters' space

No	b_c	ρ_c	h_a	h_p	b_a	b_p
	m	%	cm	cm	cm	cm
1	2.20	1.0	10	13	81	65
2	2.30	1.1	11	18	84	66
3	2.40	1.0	13	14	96	70
4	2.30	1.1	14	15	97	70
5	2.50	1.2	8	16	98	84
6	2.60	1.3	8	16	98	84
7	2.70	1.3	8	16	98	84
8	2.80	1.3	8	16	98	84
9	2.90	1.4	8	16	98	85
10	2.90	1.6	8	18	98	89
11	2.95	1.6	8	18	98	89
12	2.95	1.6	8	18	99	89
13	3.00	1.8	8	17	99	85
14	3.10	2.0	8	17	99	87
15	3.20	2.7	8	17	99	86
16	3.25	2.7	8	17	99	86

Still, using IDA considerably lengthens the computation time needed, a disadvantage that may at least be partially alleviated by ingeniously combining the fast static pushover with the slow but accurate IDA. The static pushover could quickly discern cases that do not satisfy the constraints or are not good candidates for the Pareto front, allowing us to drop them before running the more accurate analysis. Thus, IDA would be saved for the promising candidates only, achieving the accuracy of the proposed procedure at a lower cost. Additionally, advanced and faster evolution techniques together with better constraint handling schemes that take advantage of offsprings that violate them will help improve the present scheme.

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