Prescriptive Approaches in Performance-Based Design? A Case-Study on Base Isolation

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ABSTRACT: The collapse performance of code-designed base-isolated structures has recently received considerable criticism, having been found to be deficient vis-à-vis conventional buildings in several situations. As a remedy, prescriptive minima with a tenuous probabilistic justification have been recommended in the literature for the bearing deformation capacity. These are independent of structure or site characteristics, yet they are already finding use in design. We put this concept to the test by means of a case study of a seismically isolated steel structure that rests on the roof of two adjacent high-rise reinforced concrete towers. To seismically isolate the steel structure, Friction Pendulum Bearings (FPBs) are used, and their displacement capacity is determined to comply with a performance objective of 1% probability of collapse in 50 years. The case study possesses two salient features that distinguish it from pertinent past investigations. The first is that the isolated steel structure rests on top of two others and consequently it is subjected to narrow-band roof acceleration time histories, shaped by the filtering of the ground motion excitation through the supporting buildings. The second is that the two supporting towers have different modal characteristics, thus displacement demands imposed to the FPBs are mainly affected by their in-phase or out-of-phase movement. Overall, a case-specific true performance-based design is shown to achieve the desired safety while requiring 1.5 times lower displacement capacities for the bearings, when compared to prescriptive "performance-based" approaches.

1. INTRODUCTION

Base-isolated buildings have long been believed to exhibit a superior seismic performance as opposed to conventional non-isolated buildings. Quite surprisingly though, recently, they were subjected to considerable criticism (e.g. Kitayama and Constantinou, 2018a) for their performance in seismic scenarios other that the ones that were designed for, since they were found to exhibit unacceptable collapse probabilities. These were in many cases worse that those experienced by non-isolated structures (Iervolino *et al.*, 2018).

In response to this evidence, prescriptive design criteria for the displacement capacity of the isolators have been proposed, e.g. recommending that one design them for the imposed displacement demand at a (non-collapse) design intensity (e.g., at the maximum considered earthquake) times a factor. Although this approach can surely enhance seismic collapse performance, it injects an unknown safety margin to the isolated system while adding substantial costs. Interestingly, compared to conventional buildings, isolated structures are by design relatively simple to model and analyze. Therefore, an explicit consideration of risk is a viable alternative, as will be shown for the case study of a one-story base-isolated steel structure that sits on top of two structurally-independent high-rise reinforced concrete (RC) towers.

2. WHICH DESIGN METHODOLOGY?

In view of the ever-present uncertainties and the observed unsatisfactory seismic performance of base-isolated structures in past earthquake events, there is an established tendency nowadays to overdesign them. However, overdesigning a structure to guarantee its satisfactory performance in the set performance objectives is far from what an efficient performance-based design is meant to be. In fact, the real objective of performance-based design is to be conservative to allow for the uncertainties associated with the seismic hazard, material randomness and modeling inaccuracies, but this conservativeness should be limited to that required by the desirable confidence level (Vamvatsikos, 2017), which reflects a tunable safety factor reflecting the consequences stemming from the violation of specific performance objectives by the designed structure (Katsanos and Vamvatsikos, 2017).

Up until now, several seismic design methodologies have appeared in the literature. Nevertheless, the majority of them are intensity-based and consequently not fully risk-consistent (Vamvatsikos *et al.*, 2016). In intensity-based design methodologies the output is supplied in the form of an Engineering Demand Parameter (EDP, e.g. max interstory drift ratio or peak floor acceleration) evaluated at the intensity level(s) of interest, or to put it in a different way, the output provides the statistics (typically the mean) of the EDP of interest at one or more ground shaking intensity levels. In principle, an intensity-based approach it is only implicitly risk-aware, since the probability of safety (i.e. the probability of the structure exceeding a specific performance objective, e.g. collapse) is indirectly accounted via the return period of the ground motion intensity level. For instance, such methodologies imply that for a structure to satisfy the collapse performance objective (that is often paired with a 1% probability of collapse in 50 years) it is sufficient to test the structure at a design intensity with 10% probability of exceedance in 50 years and apply a safety factor. In fact, this implies that the implicit safety consideration at the intensity level directly propagates to the resulting risk, essentially ignoring the effect of response variability and the shape of the seismic hazard curve (Vamvatsikos, 2017). Finally, the design intensity spectrum is represented by a uniform hazard spectrum to which ground motion records are typically matched effectively ignoring the correlation among spectral values at different periods, and how this varies with intensity at any given site.

A more elaborate, yet of questionable effectiveness, technique is the risk-targeted spectra that were adopted in ASCE 7-10 (2010) in replacement of the uniform hazard spectra. In principle, a risk-targeted spectrum accounts both for the site seismic hazard and the probability of structural failure, aiming to offer a design basis for the intensity-based approaches that will result in uniform collapse probabilities. The methodology for obtaining a risk-targeted spectrum involves convolving the seismic hazard curve with a generic fragility curve that is deemed to be representative for the building stock in the considered country; by working backwards from a targeted 1% collapse probability in 50 years one may determine a new design spectrum which theoretically, if adopted in conventional design, will inject the required safety against collapse to any structural configuration. The use of the generic fragility functions is indeed the weak point of this approach, since apparently the variability of the building stock at the country level is difficult to depict by a single fragility. Moreover, it has been also demonstrated in the past that, even within the same building class, the variability among the vulnerability or fragility curves of the index buildings that represent it is significant (Kazantzi *et al.*, 2014). In sort, while risk-targeted spectra can largely harmonize the risk among quite different structures, which constitutes a feat of significant usefulness for conventional seismic design, they still cannot reliably target a specific risk value (Spillatura et al., 2019).

Such risk-ignorant “performance-based” design concepts are currently widely applied in the design of base-isolated systems, in the form of prescriptive formulas for the required displacement capacity of bearings, e.g., suggesting a displacement design value of at least 1.5 times the mean demand evaluated under the risk-targeted maximum considered earthquake (MCE_R), e.g., as in Kitayama and Constantinou (2018a,b). Such requirements may be simple to implement but they are not tied to the site hazard characteristics or the structural behavior of the system and thus may not deliver the levels of safety that base isolation can achieve.

By contrast, a risk-based assessment results in direct estimates of the EDP demand distribution and correspondingly the annual rate of exceeding a particular EDP level (Lin *et al.*, 2013a, b). In design terms, a risk-based approach aims at verifying for the designed structure that the risk of exceeding (i.e., violating) one or more limit-states does not exceed the target(s) specified by the code or agreed with the client (e.g. the risk of collapse not exceeding the target risk set to 1% probability of collapse in 50 years). The Mean Annual Frequency (MAF) of an EDP exceeding a specific limit-state (*LS*), denoted as λ_{LS}, may be obtained by integrating the probability of observing an *EDP* amplitude that is greater than the *LS* at a given ground motion intensity level $IM = x$ (i.e. the fragility), with the rate of observing this ground motion intensity level (i.e. seismic hazard). In a mathematical form this could be expressed as:

$$\lambda_{LS} = \int P(EDP > LS|IM = x)|d\lambda(IM = x) \quad (1)|$$

The latter approach was adopted to deliver an entire solution spectrum for the case-study at hand, thus allowing the client to make an informed decision as to which collapse risk level fits his/her needs. Thus, performance targets of 1% and 0.2% collapse probability in 50yrs will be verified by convolving the hazard with the collapse fragility corresponding to a range of different allowable deformations in the bearings.

3. BUILDING DESCRIPTION

The Six Towers project in Nicosia, Cyprus, is a building complex of six high-rise RC buildings that are separated, every two buildings, by expansions joints to essentially form three dynamically independent building units (A, B and C, see Figure 1). The lateral load resisting system of the RC structures consists primarily of shear walls, mainly arranged along the transverse direction (Global Y, or top-to-bottom in Figure 1). At the top of building units A and B, which for modeling purposes have a height of 46.95m, rests a single-story base-isolated steel structure that is 5.5m high (see Figure 2). The bearings used for isolating the steel structure are Friction Pendulum Bearings (FPBs) that according to the client's specifications have a reference radius of R_{eff} = 318" = 8077mm and a minimum friction coefficient of f = 7%. The building complex was designed according to the provisions of Eurocode 8 (EN1998-1) for seismic loading. The main seismic design parameters according to EN1998 (CEN 2004) are:

1. Soil type C
2. Importance class II (i.e., ordinary)
3. Medium Ductility Class
4. Behavior factor for concrete towers: q_x=q_y=2.76
5. Reference peak ground acceleration a_{gR} = 0.20g

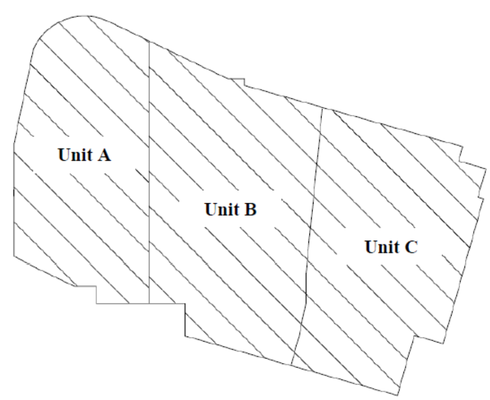


Figure 1: Plan view of the Six Towers complex showing the three dynamically independent building units.

Figure 2: Side view of the Six Towers building complex showing the three independent units and the location of the expansion/seismic joints.

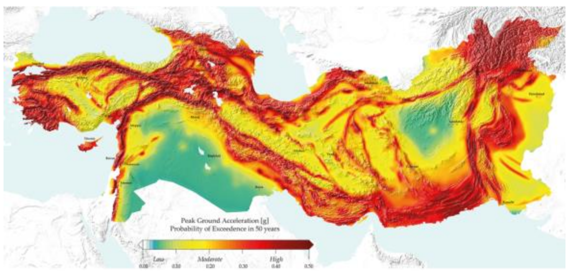
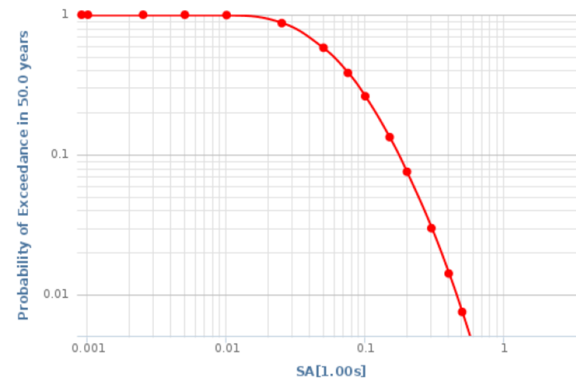


Figure 3: Reference Seismic Hazard Map of Middle East depicting PGA levels with 10% probability of exceedance in 50 years (Giardini et al., 2016).

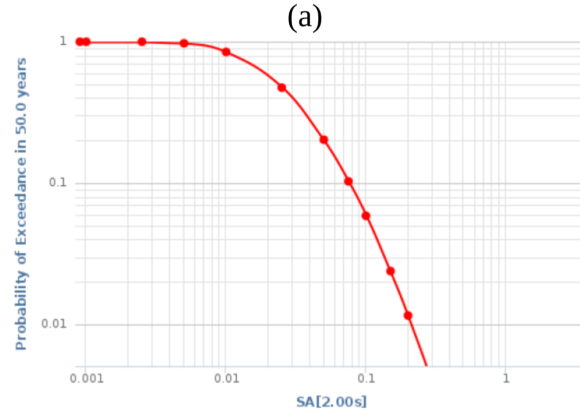
4. SITE HAZARD

Accurate performance assessment requires accurate seismic hazard estimates. At present, the best available peer-reviewed seismic source model for Cyprus is the Earthquake Model of Middle East (EMME2014) as described by Erdik *et al*. (2012). This incorporates a comprehensive model of the entire Middle East, complete with seismic mechanisms, rates, ground motion prediction equations and logic trees for handling uncertainty. The corresponding coverage via a 10% in 50yrs peak ground acceleration hazard map appears in Figure 3. A single location at longitude 33.332 and latitude 35.222 was employed to determine spectral acceleration hazard curves (Figures 4a and 4b).

It should be noted that EMME2014 results are only available for rock sites, i.e., soil type A according to EN1998 (CEN 2004). To transform from the implied bedrock to Soil C at the surface, where the foundation of the building complex lies, the spectral acceleration values were amplified by the ratio of EN1998 spectra for Soil C over those for Soil A at any given period. For the periods of interest, between 0.8sec and 2sec this resulted to amplification factors of 1.725.



(a)



(b)

Figure 4: (a) S_a(1.0s) and (b) S_a(2.0s) hazard curves for Nicosia soil type A from EMME2014.

5. STRUCTURAL MODELLING

5.1. Elastic model

For investigating the seismic performance of the base-isolated steel structure that rests on the top of the RC tower units A & B, we have employed a 3D model for the structures of interest (see Figure 5). Overall, the structural model was constructed having in mind that it should be able to depict/reproduce two salient features of the project at hand. The first is that the isolated steel structure rests on top of two others and consequently it is subjected to narrow-band roof acceleration time histories, shaped by the filtering of the ground motion excitation through the supporting buildings. The second is that the two supporting towers have different modal characteristics, thus displacement demands imposed to the FPBs are mainly affected by their in-phase or out-of-phase movement.

All analyses were undertaken using the OpenSees analysis platform (McKenna and Fenves, 2001). The structural elements were modelled as elastic beam-column elements whereas for modelling the Friction Pendulum Bearings we have employed the Single Friction Pendulum Bearing element that is readily available in the element library of OpenSees. The utilization of a 3D model as opposed to a 2D representation for the structures of interest was deemed to be more appropriate for depicting any torsional behavior under the seismic loading.

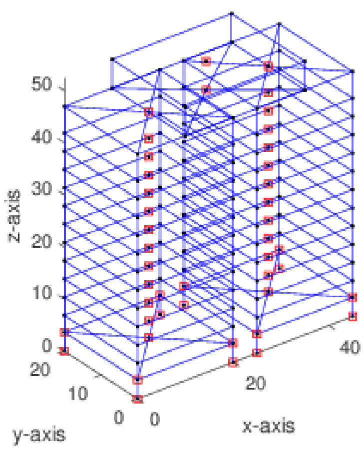


Figure 5: 3D model of Building units A & B and the base-isolated steel structure. The distance between the buildings was employed for modelling purposes and does not reflect the actual width of the expansion joint incorporated in connecting impact-springs

Rigid diaphragms were assumed at the floor levels to capture the effect of the concrete slabs. The floor mass (rotational and translational) was concentrated at the center of each floor diaphragm. The supports at the basement were assumed to be fixed whereas, the top of the basement (ground level) was restrained against lateral deformations. The lateral stiffnesses at each story in the two orthogonal directions of the modelled towers were appropriately calibrated to match the dynamic properties estimated via detailed design-grade RC tower models. A damping ratio of 5% was employed at the two fundamental modes of units A and B (Global X direction), as standard for RC structures. Note that these two modes are indeed the ones that most contribute to the FPB deformations, while the 5% value is considered reasonable for elastic models of RC buildings under severe excitations.

Potential pounding due to closing of the seismic gap separating the towers was modeled by incorporating appropriate elastic perfectly-plastic gap elements with an allowable compression gap of 0.5m. A relatively soft impact-spring stiffness was employed to avoid numerical instabilities, while only allowing sub-centimeter intrusion of one building into the other.

With reference to the three structures, we restricted the modeling/analyses to the realm of the elastic behavior since the two RC units and the steel structure are deemed to be flexible and ductile enough for the equal displacement rule to hold. We expect some reduction of the narrowbandendness of the roof motions due to the inelasticity of the RC structures. This has been shown to considerably reduce any resonance effect, so it is expected to act beneficially or at least neutrally where no resonance is present, but certainly not against the conservativeness of the assessment.

5.2. Modal characteristics

Given the mass of each floor, the stiffness of the building units was appropriately calibrated, so that the reduced-order OpenSees model eigenvalues match as much as possible those provided by the detailed models. The results are summarized in Table 1. The evaluated mode shapes also demonstrate that, potential torsional behavior with reference to the steel base-isolated unit is appropriately captured.

5.3. Dynamic behavior

Building units A and B have a very distinctive behavior. Due to the transverse arrangement of the shear walls, they are very stiff in the transverse direction (Global Y) while remaining relatively soft in the longitudinal direction (Global X). Thus, most of the deformation appears along the seismic joint, with some pounding actually happening for very intense ground motions. Due to the relatively soft bearings, the top steel structure largely retains its initial position and the rooftops of units A and B move below it, mainly along the longitudinal axis. It is this deformation of A and B that drives the bearing displacement, especially when they move away from each other.

Table 1: The first 12 mode of vibration evaluated using the 3D simplified OpenSees model. The number after the axis designation of X or Y reflects the hierarchy of the mode in the given direction. The fundamental modes for each unit & direction are shaded grey and the second modes orange.

Mode	T [s]	Unit/Mode Description
1	15.641	Steel/X
2	15.637	Steel/Y
3	2.114	Unit A/X1
4	1.144	Unit B/X1
5	0.856	Unit B/Y1 Steel/Torsion
6	0.769	Unit A/X2
7	0.582	Unit A/Y1 Steel/Torsion
8	0.520	Unit A/X3
9	0.462	Steel/Torsion
10	0.452	Unit B/X2
11	0.369	Unit A/X4
12	0.314	Unit B/Y2 Steel/Torsion

6. SEISMIC PERFORMANCE ASSESSMENT

6.1. Collapse fragility assessment

Nonlinear response history analyses were performed at both a moderate and a high intensity level by employing two ground motion sets that were carefully selected for European sites of moderate and high seismicity (Kohrangi and Vamvatsikos, 2016), each comprising of 30 ground motion records times two horizontal components each. Spectral acceleration at a single period was the intensity measure of choice. Due to the flexible few first modes of Building unit A contributing most of the deformation demand, this period was chosen to be the 60/40 weighted mean of the first two structural modes, i.e., 2.114s, and 1.144s yielding a rounded value of T=1.75s. The discrete data points in terms of the 5% damped spectral acceleration (geometric mean of the two horizontal components) and the maximum recorded FPB deformation were fitted via a power law regression line (Cornell *et al.*, 2002), as shown in Figure 6.

The results were employed to produce collapse fragility curves for a range of different FPB deformation capacities spanning between 0.3m and 1.5m, as shown for example in Figure 7. An additional epistemic uncertainty dispersion (log-standard deviation) of 0.20 was assumed to infuse additional conservativeness. The overall collapse capacity dispersion is 51%.

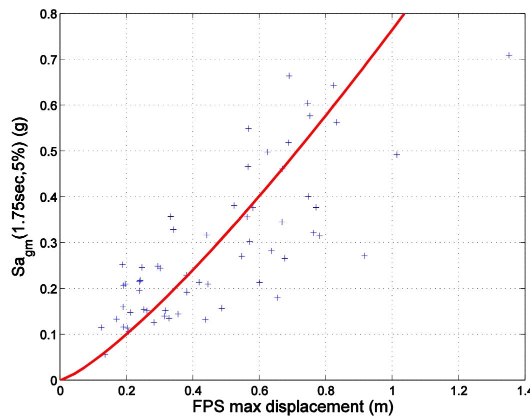


Figure 6: Results from 60 nonlinear response history analyses in terms of the maximum FPB displacement versus the geomean value of Sa(1.75sec) at 5% damping. The red line represents the power law regression on the cloud of points.

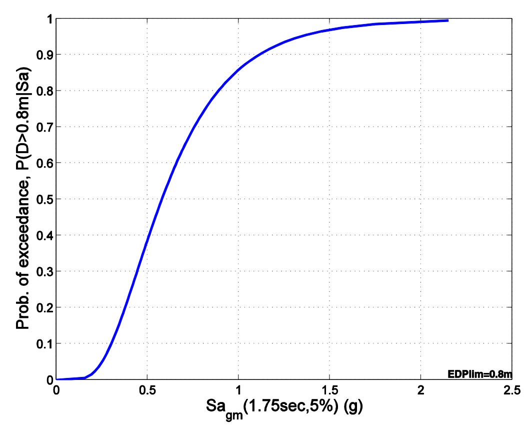


Figure 7: The collapse fragility curve for the base isolation system when the bearings have a maximum deformation capacity of 0.82m.

6.2. Maximum displacement risk curve

The hazard curve determined for S_a(1.75sec) (geomean component, 5% damped) was convolved with the collapse fragilities estimated for the FPBs. The resulting collapse risk curve appears in Figure 8. Based on these results, the following three options are delineated:

(a) Option 0: Employ EN15129 (CEN, 2009) and design the FBPs for a displacement capacity of 1.5 times the Design Level Earthquake (or 10% in 50yrs) demand of 30cm, i.e, 1.5×30cm = 45cm. According to Figure 8, this will achieve a collapse risk of 5.2% in 50yrs. This is considered unacceptable when compared to the implied safety of EN1998 designed non-isolated structures and it constitutes a clear failure of the EN15129 standard to provide adequate safety. This option is not recommended.

(b) Option 1: Target a collapse risk of 1% in 50 years. Then a bearing displacement capacity of 0.82m is required. This fully complies with the stated targets of ASCE/SEI 7-10 (2013) for ordinary structures (Importance Class II or Risk Category II) and it approximately complies with the *average* implied safety of EN1998 conventional structures (Iervolino *et al*., 2018). This is considered the best compromise between cost and safety for this building, assuming installation of the enlarged isolators is possible given the available space at the rooftops of Buildings A and B. The introduction of fail-safe mechanisms (e.g., perimeter stoppers) can even further improve performance under rare ground motions.

(c) Option 2: Comply with SISCF standard requirements (Zayas *et al*., 2017) and target an improved collapse risk of 0.2% in 50yrs. This requires a FPB displacement capacity of 1.31m according to Figure 8. As recent results (Iervolino *et al*., 2018; Spillatura, 2018) from the RINTC project have shown, most EN1998-conforming structures (other than EN15129 isolated buildings and precast structures) have collapse risks less than 0.5% in 50 years, with this value increasing for regions of higher seismicity and buildings of larger height. This in fact supports increasing the safety of the isolation system to lower the collapse risk. Still, at such large intensities, the uncertainties involved in the near-collapse behavior of the supporting structures are too large to ignore and thus there is not sufficient confidence in the behavior of the two concrete towers to support an even safer base-isolated top story. Therefore, the increased capacity of the bearings would be best taken advantage of by a redesign of Building Units A and B to a higher Importance Class.

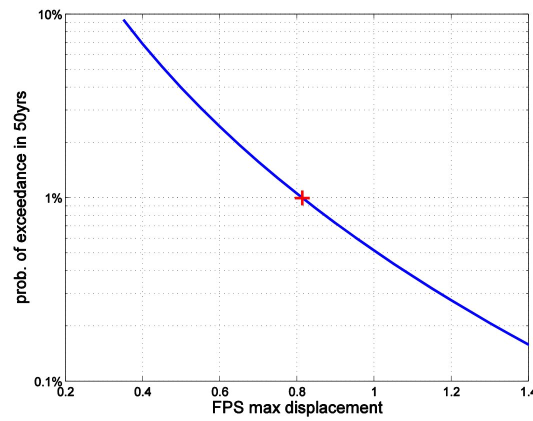


Figure 8: The FP bearing maximum displacement risk curve in terms of the PoE in 50yrs. The value of 0.82m corresponding to the 1% probability of exceedance is indicated by a red cross.

7. CONCLUSIONS

An explicit performance-based design methodology was demonstrated by means of a case study that involves a base-isolated building. The presented design methodology is believed to be superior over prescriptive performance-based approaches since: (a) it yields risk-consistent structural design; (b) it does not inject unnecessary conservatism and hence results in more economical final design products and (c) the collapse risk or any other targeted performance objective can be precisely tailored to the client needs. Prescriptive intensity-based design approaches may be easier to implement, but they simply fail to reach such standards of quality.

8. ACKNOWLEDGMENT

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