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Performance-based seismic design of a modular pipe-rack

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Abstract

Aimed at demonstrating the benefits of using a robust PBD (performance-based design) framework in the engineering construction industry, the seismic analysis of a typical pipe-rack module is presented in this paper, comparing prescriptive and performance-based approaches. The case-study steel frame is 6 m long, 8 m wide and 10 m tall, and is representative of this type of structures in the oil and gas industry. The hazard analysis is used to select a representative set of recorded accelerograms for increasing values of the seismic intensity measure (IM), chosen as the spectral ordinate at the fundamental period of vibration of the structure. Nonlinear time-history analyses are carried out with the commercial software SAP2000 to establish the fragility curves relevant to the pipe rack. The process is automated through MATLAB coding and a range of $EDPs$ (engineering demand parameters) are statistically characterised, namely internal forces, deformations and absolute accelerations, which in turn are associated with various DMs (damage measures).

Keywords: performance-based design; fragility curves; seismic analysis; modular structures; pipe-rack;

1. Introduction

Modularisation is becoming progressively popular in recent years, especially in sectors such as power stations and the oil and gas industry. Indeed, many of the facilities related to these fields are located in remote areas often characterised by lack of skilled workforce and harsh climatic conditions. The current seismic design methods for modular structures are mainly focused on structural resistance and require an acceptable demand-to-capacity ratio (D/C), providing life safety and damage control through the application of multiple safety factors. This strategy appears to provide an adequate level of earthquake protection from the point of view of safety of life. However, it can often lead either to over-conservative solutions and, consequently, more expensive construction projects, or, in certain situations, to have disproportionate economic losses caused by structural damage and the loss of use of the facilities [**?**]. Considering the ever-growing importance of modularisation in the construction and industrial fields, many researchers have highlighted the inefficiency of the current design methods and tried to apply new seismic approaches [**? ? ?**]. Differently to prescriptive methods, the goal of performance-based design (PBD) is to achieve a set level of performance during the lifespan of the structure, which can be directly related to appropriate consequences; in turn, the latter can be quantified in different ways, including monetary cost, considering both initial investments and likely maintenance costs [**? ?**].

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2. Performance-Based Design

2.1. PEER Formulation

Because of the limitations related to the first generation of performance-based earthquake engineering (PBEE), since the early 2000s the Pacific Earthquake Engineering Research (PEER) centre started developing a new PBEE methodology [?]. The innovative key feature of the PEER's PBEE approach is that the performance is rigorously calculated in a probabilistic manner. The framework consists of four successive stages, namely: Hazard Analysis, Structural Analysis, Damage Analysis and Loss Analysis. The quantitative data delivered by the combined analyses allow the decision-makers to identify the "optimal" solution, in whichever sense is most appropriate for each particular design. The framework can be mathematically expressed as **??** [?]:

$$p[DV|O, D] = \int\int\int p[DV|DM] \cdot p[DM|EDP] \cdot p[EDP|IM] \cdot p[IM|O, D] \cdot dIM \cdot dEDP \cdot dDM, \tag{1}$$

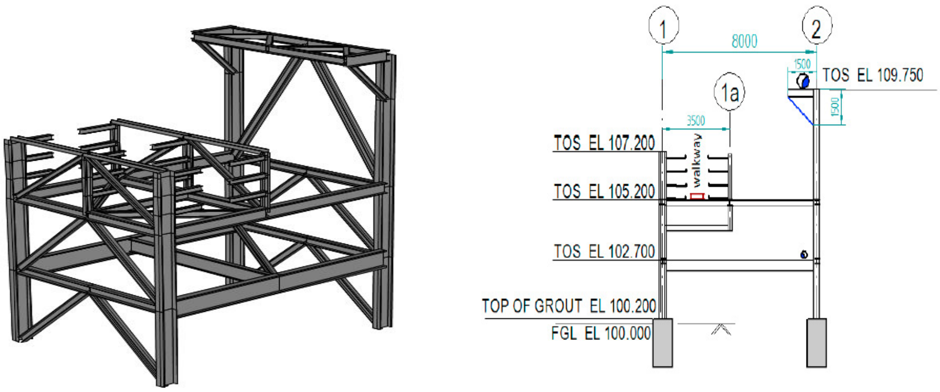
where:

$p[X]$ = probability density function (PDF) of the random variable X; $p[X|\ Y]$ = conditional PDF (CPDF) of X given the event of $Y = y$; O = Location of the structure; D = design of the structure; IM = Intensity measure of earthquake site effects; EDP = Engineering demand parameter as a measure of the structural response; DM = Measure of any physical damage; DV = Decision variable, that is the performance parameter of interest.

3. Application, Results and Discussion

3.1. Case Study

The steel frame considered in this study (Figure **??**a) has been adapted from an actual structure designed for an industrial plant in a moderate seismic region. It consists of a pre-assembled rack (PAR) without loose items, and is 6 m long, 8 m wide and 10 m tall. The rack is used for supporting process pipes and electrical trays at different levels of elevation (EL) (Figure **??**b). The structure is made of hot rolled sections of ASTM A572 grade 50 steel, with thick-plate girders, which make the structure quite stiff (its fundamental period of vibration in the transverse direction is $T_1 = 0.2$).



(a) (b)

Fig. 1: PAR: (a) axxonometric view, (b) elevation view.

The study considers two different models: the original case-study module (this will be referred to as "structure A") and a modified module ("structure B"), obtained by considering thinner sections for beams and columns in order to have a less stiff PAR ($T_1 = 0.9$). The responses of the two structures are compared considering both a prescriptive and a PBD approach.

3.2. Response Spectrum Analyses

Before proceeding with the PBD, the response spectrum method was used to verify the seismic-code compliance of structures A and B. The analyses were performed with SAP2000 [**?**] and the design spectrum was built according to the international building code (IBC) [**?**]. In order to have the values of spectral acceleration, S_a, corresponding to each period, T, the parameters of the spectrum were obtained by introducing the coordinates of a site in California and by considering a site class B corresponding to "firm rock".

3.3. Performance-Based Design Approach

3.3.1. Hazard Analysis

The probabilistic seismic hazard, $p[IM|O, D]$, was developed considering all the parameters related to the location and the design variables of the case-study structure, including magnitude, faults and soil conditions. Spectral acceleration at the period of the first mode, $S_a(T_1)$, was considered as IM and the hazard curves, built with a dedicated software [**?**], were expressed in the form of $S_a(T_1)$ against probability of exceedance (PoE) in 50 years (Figure **??**). On both the hazard curves, ten groups were defined with 10% increments of the PoE (whose midpoints are marked with thick dots in Figure **??**).

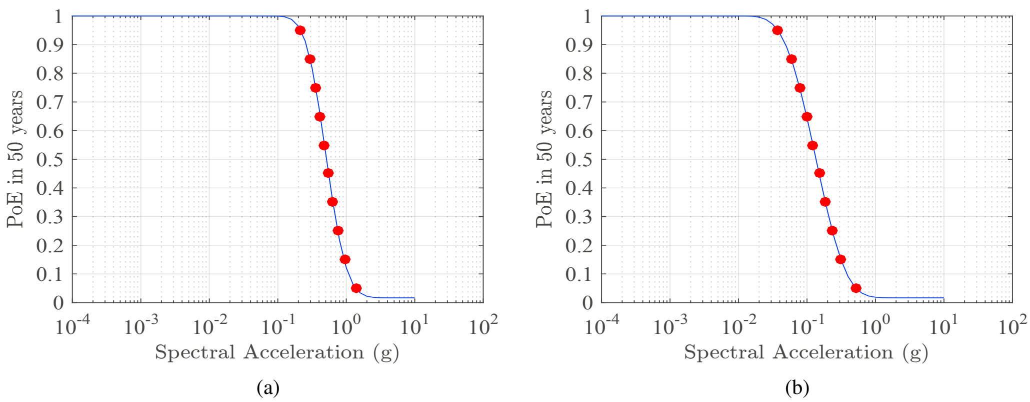


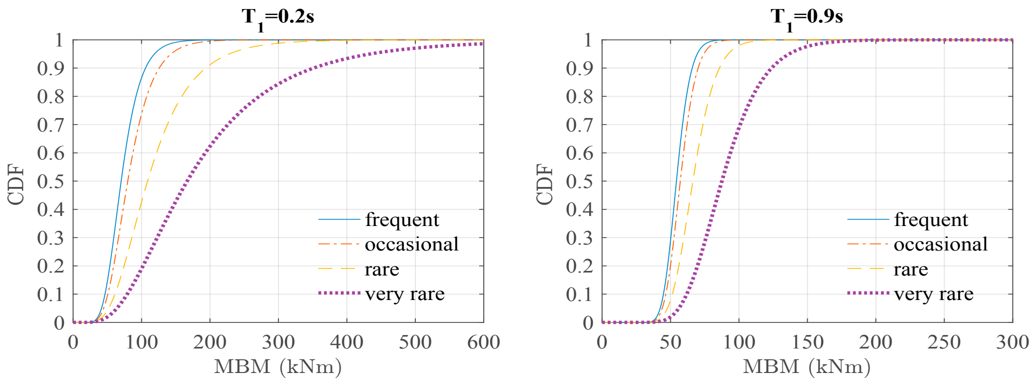
Fig. 2: Hazard curves: (a) Structure A, (b) Structure B.

3.3.2. Structural Analysis

The probabilistic characterisation of the structural response, $p[EDP|IM]$, was achieved for the ten intervals defined in the previous stage in terms of two damageable groups, namely structural and non-structural components. The maximum bending moment (MBM) of the first floor beams and the peak absolute accelerations (PAA) of a single-degree-of-freedom (SDoF) oscillator of period T_1 attached to the free end of the cantilever beams supporting the pipes were considered as relevant $EDPs$ (the acceleration was studied considering three values of the attachment's damping factor, namely $\zeta = 1\%$, 2% and 5%). After creating a database of 150 accelerograms recorded in California on sites of class B, the best seven in terms of scaling factor for each group were used to perform non-linear time history analyses in SAP2000, allowing the formation of plastic hinges. The required $EDPs$ were then post-processed in MATLAB [**?**] and the parameters of lognormal distributions were calculated for each IM level. To do this, median and 90th percentile of all the distributions were computed, the data were fitted by quadratic or linear relationships (whichever provided the best fit), and extrapolation was only used for the "very rare" design scenario (corresponding to 2% of PoE in 50 years). Cumulative distribution functions (CDFs) for values of $S_a(T_1)$ corresponding to four damage levels [**?**] were obtained from the assumed statistical distribution (Table **??**). Figures **??** to **??** reveal that both the bending moments and the absolute accelerations in the structure A have larger values and a wider dispersion.

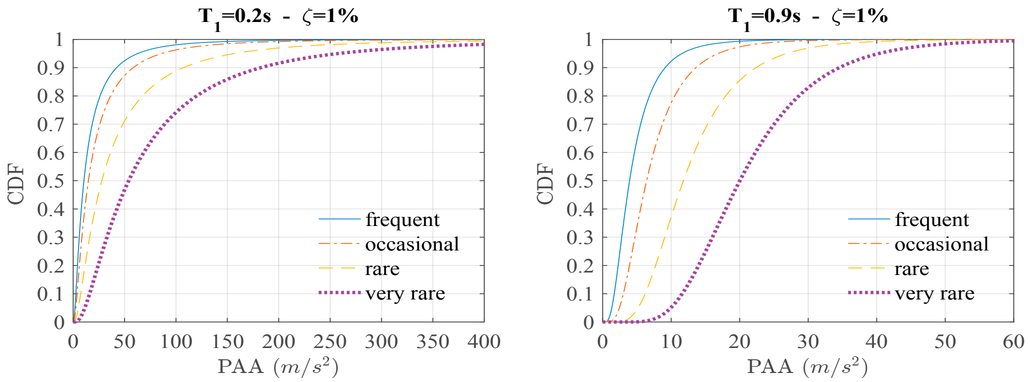
Damage level	Frequent	Occasional	Rare	Very Rare
PoE$_{50}$ [%]	87	50	9.5	2.0
Sa case A [g]	0.30	0.50	1.10	2.30
Sa case B [g]	0.06	0.14	0.40	1.00

Table 1: PoE$_{50}$ of damage levels and corresponding Sa on the hazard curves.



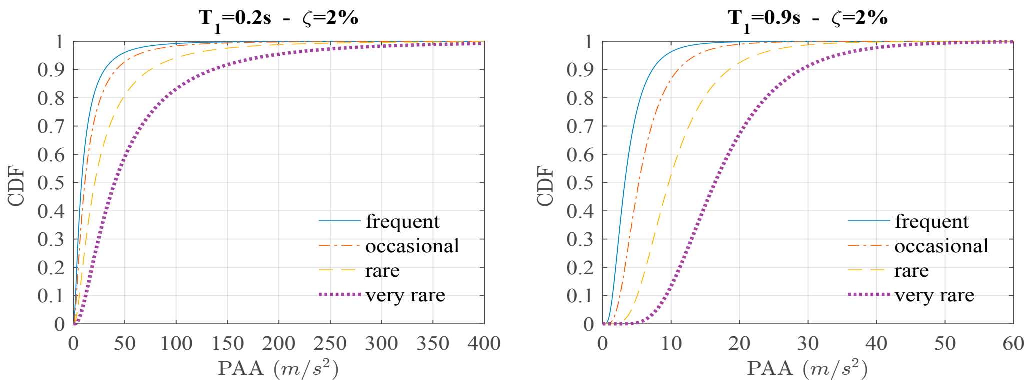
(a) (b)

Fig. 3: CDF of MBM: (a) Structure A, (b) Structure B.



(a) (b)

Fig. 4: CDF of PAA with ζ=1%: (a) Structure A, (b) Structure B.

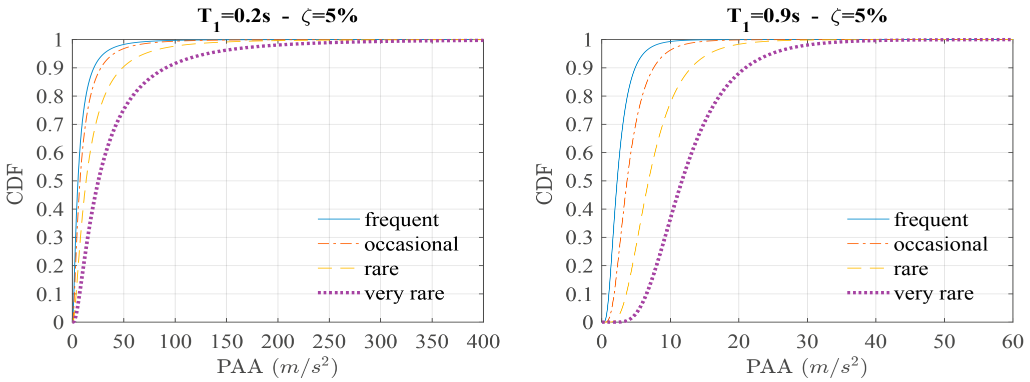


(a) (b)

Fig. 5: CDF of PAA with ζ=2%: (a) Structure A, (b) Structure B.

3.3.3. Damage Analysis

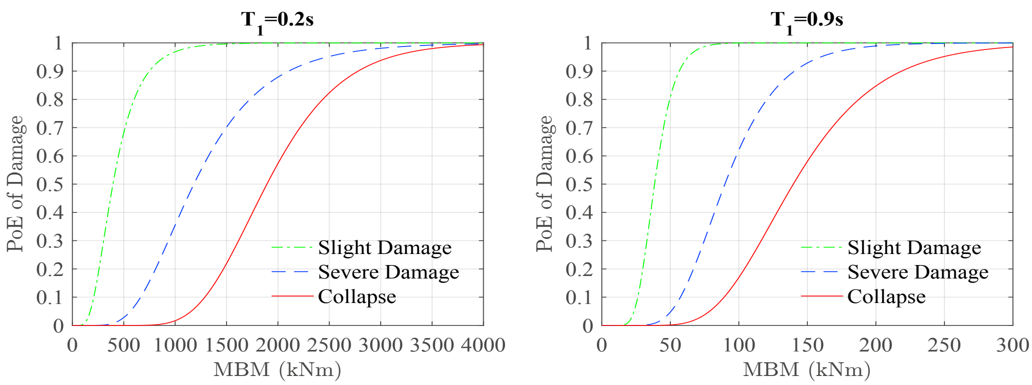
In this stage of the PBD, the CPDF, $p[DM|EDP]$, is evaluated to relate the $EDPs$ to the DMs, which in turn describe the physical damage to a facility. In order to do that, a set of fragility functions of both $EDPs$ was developed.



(a) (b)

Fig. 6: CDF of PAA with ζ=5%: (a) Structure A, (b) Structure B.

Regarding the MBM, three levels of damage were identified: slight, moderate and collapse, corresponding to different states of the plastic hinges when the structure is subjected to a pushover analysis. In particular, the first level is reached when the structure plasticises for the first time; the second when the plastic rotations preclude the immediate occupancy condition, as defined by [**?**]; the third when the structure is on the verge of collapse.



(a) (b)

Fig. 7: Fragility curves for structural components: (a) Structure A, (b) Structure B.

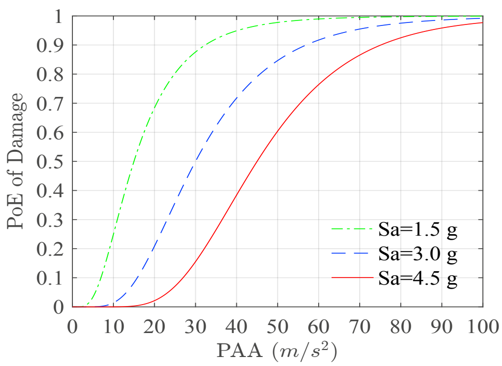


Fig. 8: Fragility curves for non-structural components

Interestingly, the fragility curves of structure A (Figure **??** a) show that this remains in the elastic range for values of the bending moment equal to those of a very rare event, so that plasticisation is never reached in any of the analyses. On the other hand, structure B starts plasticising at relatively low values of the bending moment, without compromising the resistance of the structure against collapse (Figure **??** b). Regarding the PAA, three damage levels associated with thresholds of 1.5 g, 3 g and 4.5 g were set (Figure **??**). Only structure B meets these thresholds, thanks to its more flexible behaviour.

3.4. Discussion

The results of our investigations show that both models A and B meet the criteria of the response spectrum analysis, i.e. the prescriptive approach, but they provide different levels of performance when investigated through a PBD approach. Indeed, choosing the state of the plastic hinges and the PAA of the beams supporting the pipes as control parameters, structure A showed very over-conservative behaviour towards the plasticity and collapse as well as very high accelerations, which could cause damage to any attachment. Conversely, structure B, although less resistant, i.e. likely to experience some plastic deformations under "rare" and "vary rare" events, was characterised by more balanced structural behaviour. It is therefore clear how the information resulting from the application of prescriptive methods is limited (actually, meaningless for the relevant stakeholders, e.g. the owners of the industrial facility) and does not allow the designer to judge whether the analysed structure is actually the most suitable from the point of view of the overall costs.

4. Conclusions

Traditional prescriptive design and PBD differ in the fact that, while in the former the level of seismic risk and the acceptable level of damage are determined by the design codes, in the PBD these levels are determined taking into account the desired performance required during the expected lifespan of the structure and considering in a rational manner all the relevant sources of uncertainty [**?**]. This paper compares these two design approaches within the earthquake engineering context, highlighting the benefits deriving from the use of the PBD method on a realistic case-study structure for the oil and gas industry. The results show that using a full PBD approach in the seismic design of modular structures can lead to safer solutions with fewer uncertainties in terms of costs for the relevant stakeholders. Starting from the fragility curves, loss curves that quantify the costs of each solution could be developed, allowing the stakeholders to make decisions on the basis of quantified data. In addition, the results obtained for the seismic hazard could be used alongside the application of PBD for other sources of hazard (e.g. wind, fire, blast) and then combine all of them in order to achieve a multi-hazard design that considers all the threats that an industrial module can face during its operational life and all the possible consequences associated with them.