Seismic Fragility Assessment of Transmission Towers via Performance-based Analysis

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SUMMARY:
The knowledge and assessment of the seismic fragility curve is important to evaluate the integrity and reliability of transmission towers. In this paper, the seismic capacity assessment of the transmission tower is performed within a probabilistic frame, through a nonlinear buckling analysis and nonlinear dynamic analysis, considering the internal uncertainty of the tower and the randomness of ground motion. The performance limits of different damage states of transmission towers are determined. Finally, the seismic fragility curve of the transmission tower is evaluated by numerical Monte Carlo simulation. By the seismic fragility curve, the failure probability of the transmission tower under different magnitudes of earthquake can be visually predicted.
Keywords: Seismic, transmission towers, nonlinear buckling analysis, fragility curve, Monte Carlo simulations

1. INTRODUCTION

Overhead transmission lines play an important role in the operation of a reliable electric power system. Transmission towers are the vital components providing the supporters of high-voltage power lines. Many intensive earthquakes have happened in China recently, such as Jiji earthquake in 1999 and Wenchuan earthquake in 2008, which caused a great loss of electric power system. Failure of transmission tower under extremely intense earthquake has been reported in the literature. Therefore, it’s very imperative to evaluate the seismic risk of these towers for seismic retrofit and seismic mitigation planning. The accurate prediction of tower failure is very important for the reliability and safety evaluation of the power transmission system (Li, 2009).

The seismic risk analysis includes three contents: seismic hazard analysis, fragility analysis and earthquake-induced loss estimation. Among them, the fragility analysis is to study the probability of structural failure for a given ground motion level, and can predict probabilities of the occurrence of different damage states induced by different magnitudes of earthquakes. One of the first applications of seismic fragility analysis in civil engineering was in the report ATC-13 submitted by Applied Technology Council in USA (ATC, 1985). HAZUS developed under Federal Emergency Management Agency (FEMA) sponsorship, which is the famous program for loss estimation, incorporates fragilities for 36 categories of building and four damage states. But both of them are not entirely quantitative models, some qualitative evaluation relies on expert opinion to a considerable degree. Recently, the emerging methodologies depend more on computation efforts, in other words, the trend of fragility analysis is shifting from qualitative paradigm toward quantitative paradigm.

In this paper, the seismic capacity of transmission tower is evaluated by using nonlinear buckling analysis method and nonlinear dynamic analysis, considering the inherent uncertainty of the structure and ground motion. And the performance limits of different damage states induced by earthquake are determined. The objective of this literature is to evaluate the fragility curve of transmission towers based on seismic performance analysis considering the inelastic structural behaviour and the uncertainties.

2. PROBABILITY-BASED SEISMIC PERFORMANCE ANALYSIS

The inelastic behaviour of the transmission towers subjected to the extreme earthquakes has been investigated extensively. The nonlinear static pushover analysis and incremental dynamic analysis (IDA) method (Vamvatsikos and Cornell, 2002) have been widely used in earthquake engineering for evaluating the structural capacity curves considering seismic excitations. The towers might collapse or be damaged when shaken by intensive earthquakes. However, the information relating the nonlinear inelastic responses of such towers under extreme seismic loading with the damage severity is lacking, and the damage state of the tower remains unclear. Therefore, one of the objectives in this paper is to define the damage states of such structures under earthquake loading based on the performance analysis.

Structural seismic performance is the structural capacity to resist seismic loading, including load-bearing capacity, deformation capacity and energy dissipation capacity and so on. In this paper, the seismic capacity is described in terms of structural deformation. The key issue is to obtain the characteristic performance indices of the transmission towers under various seismic excitations in order to define different damage levels. Considering the uncertainties of the member manufacturing and environmental condition, the performance index is discretely distributed rather than deterministic. Nonlinear buckling analysis and dynamic analysis are carried out to acquire the performance of the transmission tower which is subjected to horizontal seismic loading, and Monte Carlo (MC) method is utilized for simulation of the changing environment and property variation of the structural material. Based on a sufficient number of simulations, the probabilistic relationship between seismic intensity and structural response of the tower (in terms of maximum deformation of tower) can be determined.

2.1. Tower description and finite element model

The nonlinear finite element analysis program ANSYS® is utilized in this paper for evaluating the performance of the space frame, considering the material nonlinearity and geometric nonlinearity of towers. For the numerical analysis, a lattice steel tower is considered as shown in Fig. 1. The tower has a total height of 100.6 m with an 18.75 m $\times$ 18.75 m square base area. The leg and diagonal members in the tower are steel pipes and the bracing members are steel bars with L-shape. Modelling the tower members using beam elements provides better numerical accuracy of nonlinear responses than those using truss elements. Each member of the transmission tower is modelled by beam elements (BEAM188 in ANSYS), which is based on Timoshenko beam theory considering shear deformation effects. The elasto-plastic property of the steel material is represented by an bilinear kinematic model, with the elastic modulus of 2.1×10^5MPa up to yield and 790MPa after yielding. The yield strength for leg and diagonal members and the bracing members is 345MPa and 235MPa, respectively. The finite element (FE) model consists of 505 nodes and 1378 elements, which is shown in Fig.1. The analysis of idealized configuration models is performed to obtain the pre-ultimate behaviour and the limit loads of the transmission tower without considering the coupling effect of the conduct lines.

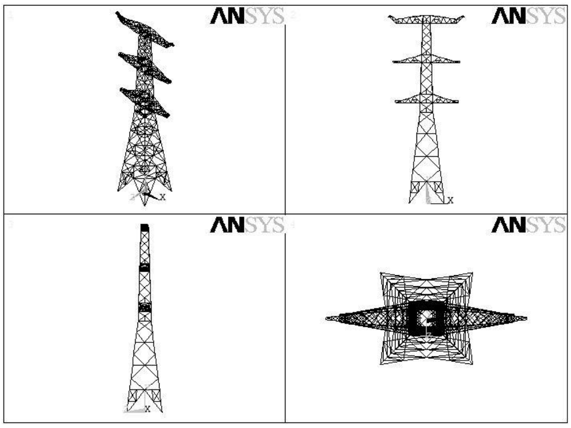


Figure 1. The finite element model of transmission tower

2.2.The nonlinear buckling analysis and the definition of damage states

The transmission towers are space steel structures and in many cases steel structures fail due to instability. Prasad Rao et al. (2010) present different types of premature failures observed in full-scale testing. Different types of failures are modelled using finite element software and the analytical results and the test results are compared with various code provisions. It is concluded that many failures of towers are caused by buckling of compression leg or bracing members and it is possible to predict the probable structural capacity of the tower by finite element non-linear analysis. Albermani et al. (1992) presented a non-linear analytical method accounting for both material and geometric non-linearity to predict transmission tower failure.

In this study, nonlinear buckling analysis considering material and geometrical nonlinearity is adopted to determine the limit loads of the tower. With a geometrically nonlinear analysis, the stiffness matrix of the structure is automatically updated to incorporate deformations which affect the structural behaviour. In ANSYS, the procedure for nonlinear buckling analysis is simple: it gradually increases the applied load until the structure becomes unstable (ie. a very small increase of the load will cause very large deflection of the structure). The nonlinear analysis incorporates the modelling of geometric imperfections, load perturbations, material nonlinearities and so on. Imperfection such as eccentric loads or initially deformed shape is introduced to perform the nonlinear buckling analysis. Firstly, eigenvalue buckling analysis (linear) is performed to predict the theoretical buckling strength of an ideal elastic structure and acquire the buckling mode shape. Then nonlinear buckling analysis is conducted after updating the geometric information of the finite element model based on the previous results of linear analysis.

The suggested earthquake load is calibrated based on the assumption that the load pattern is unchanged during an earthquake event. According to the Code for seismic design of buildings in China, an inverted triangle pattern is used to compute the lateral seismic load:

$$F_i = \frac{G_i H_i}{\sum_{j=1}^{n} G_j H_j} F_{EK} (1-\delta_n)(i=1,2,...n) \tag{1}$$

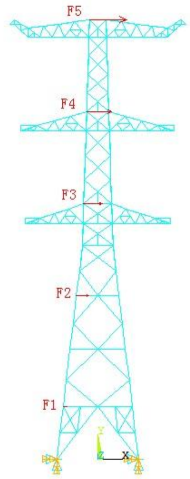
where F_i is lateral load of each segment, F_{EK} is characteristic value of seismic load, G is representative value of gravity load, H is the height, δ_n is seismic coefficient at the top of the tower.

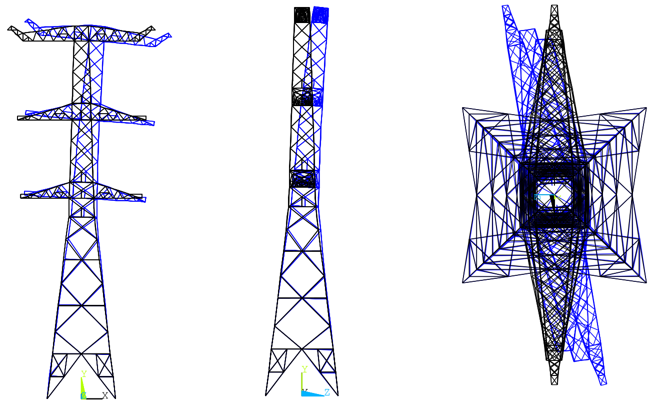
Considering the structural characteristics, the tower is divided into five segments and the seismic load acting on each segment is calculated using Eq. (1). The inverted triangle seismic load is imposed on each segment of the tower, which is illustrated in Fig. 2. The modal information from an eigenvalue analysis of the tower is listed in Table 1 and mode shapes in X direction are illustrated in Fig. 3.

Table 1. Natural frequencies and mode shapes of transmission tower in X direction

Mode	Frequency (Hz)	Mode shape
1	1.1837	First bending mode in lateral direction
2	1.2071	First bending mode in longitudinal direction
3	1.7450	First torsion mode

An eigenvalue buckling analysis is first used to determine the theoretical limit load and the bucking mode shape of the structure. By gradually increasing the lateral seismic load, a nonlinear buckling analysis is performed to predict the failure mode of the structure. Fig. 4 illustrates the buckling mode shape which is most likely to occur. It shows that the tower failure occurs due to the out-of-plane instability of the compressed leg and bracing members, which is similar to the failure modes in literature (Prasad Rao et al., 2010).





(1) (2) (3)

Figure 2. The horizontal load pattern **Figure 3.** Mode shapes

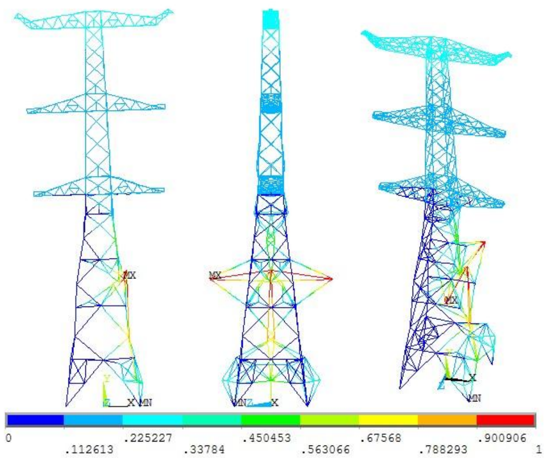


Figure 4. The first buckling mode shape

Fig.5 gives capacity curve of the tower represented by the loading-deformation curve: the total base shear force versus the top rotation angle (horizontal displacement/height, RDA) of the tower. The rotation angle of the top node increases steadily with the increasing seismic load before the base shear force reaches 2.9×10^{6}N. After that, the tower experiences rapid and large deformation with the small increase of the loading. The simulation program finally halts due to excessive deformation of the tower, which indicates the instability happens causing the structural failure. Apparently, the structural instability happens to the tower at the base shear force of 2.9×10^{6}N. After the capacity curve is determined, the elastic displacement limit, yielding displacement limit and the ultimate displacement limit of the transmission tower can be determined from the results. Then in the following step, it is possible to define the damage state of the transmission tower according to the capacity of the tower.

In this paper, three damage states are defined: minor damage, major damage, and collapse state. The ultimate rotation angle (RDA) of the tower top is defined as RDA_{co} of the collapse state, when the deformation response of the tower is greater than this value, $RDA \geq RDA_{co}$, the tower is in the state of collapse. When the tower begins to yield, the RDA of the tower top is defined as RDA_{ma}, the displacement limit of the major damage state. When $RDA_{ma} \leq RDA \leq RDA_{co}$, the tower is in the major damage state. For the minor damage state is less critical and more ambiguous than the other two states, the authors define $0.5RDA_{ma}$ as the displacement limit of minor damage state (RDA_{mi}).

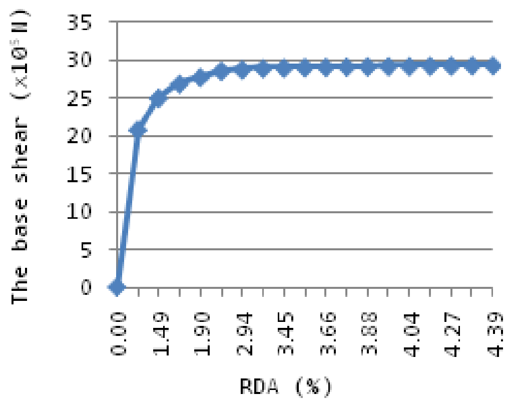


Figure 5. The base shear force versus the top rotation angle

2.3. Probabilistic analysis

In structural engineering, there exist all kinds of uncertainties, which are the inherent characteristics of the nature. The seismic performance of the structure is not deterministic but stochastic due to the uncertainties of the structure and environment. In this paper, the uncertainties of the structure are represented in terms of stochastic variables of material properties and geometric parameters, such as elastic modulus, yielding strength, passion ratio, density and the dimension of member section. In ANSYS, Probabilistic Design System (PDS) can produce probability distribution function of the aforementioned stochastic variables by Monte Carlo sampling methods. These stochastic variables are assumed to obey Gaussian distribution by setting the design value as its mean and 5% of its mean as its standard deviation. For example, the sampling of the elastic modulus of steel is illustrated in Fig. 6.

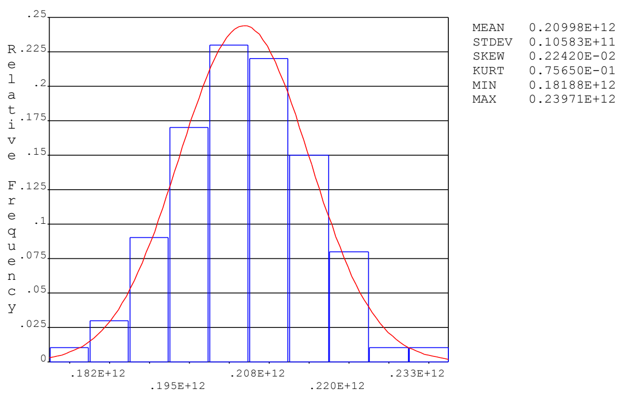
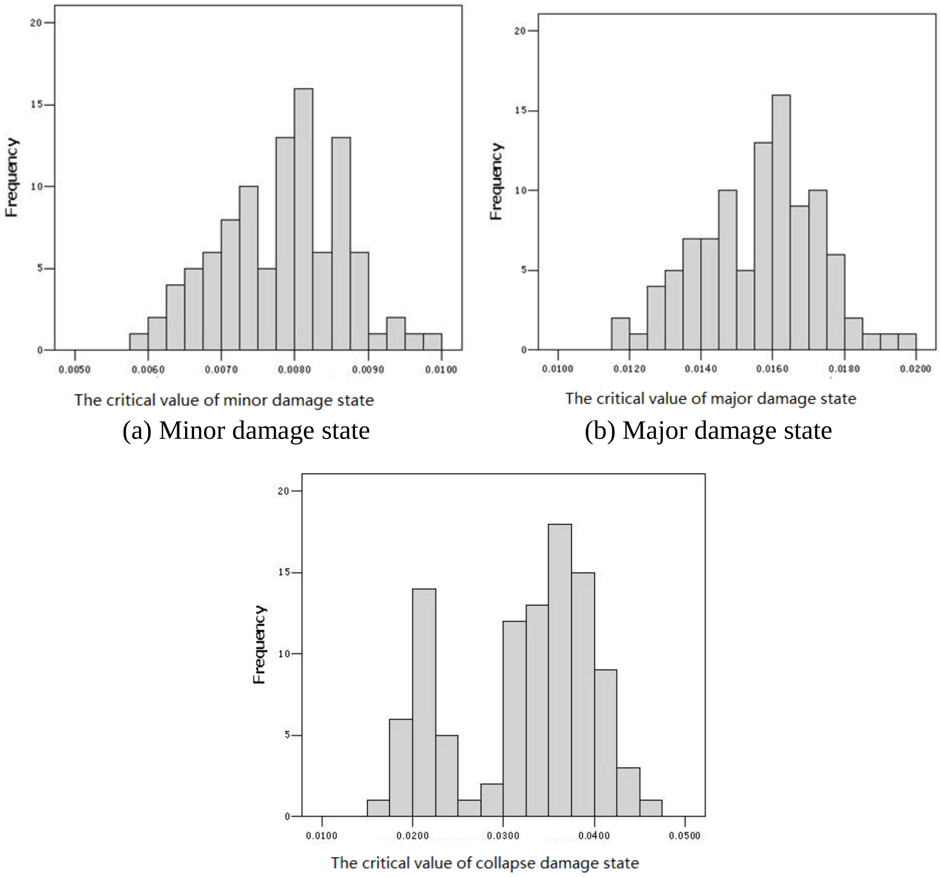


Figure 6. The sampling distribution of elastic modulus of steel

Taking the advantage of Monte Carlo (MC) methods, nonlinear buckling analysis is carried out for each sampling of the stochastic variables, and for each simulation the capacity curve is obtained. After acquiring sufficient data of simulation, displacement limits of minor damage state, major damage state and collapse state can be determined. The results are analyzed by using statistical tools, and statistical histogram of the displacement limits of three damage states are plotted in Fig.7.



(a) Minor damage state (b) Major damage state

(c) Collapse damage state

Figure 7. Statistical histogram of the displacement limits (rad)

Fig. 7 clearly shows that the displacement limits of minor damage state and major damage state is well distributed, while that of collapse state is discretely distributed with double peaks. This demonstrates that the uncertainty increases greatly when structure is approaching the collapse limit, and the capacity of the transmission tower is unstable. Two-parameter lognormal distribution functions are used to represent the displacement limit of each damage state. From regression analysis of the data, the results are listed in Table 2.

Table 2. Lognormal distribution parameter of displacement limits

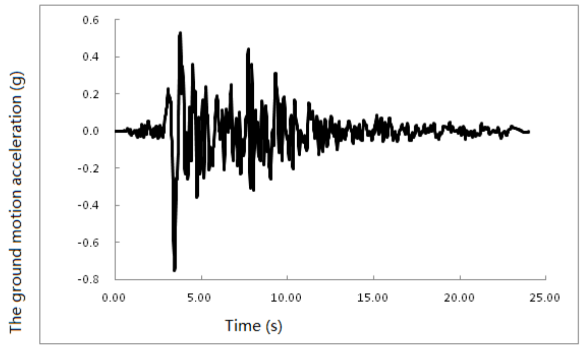
Level	Mean value of Logarithm (rad)	standard deviation of Logarithm (rad)	Expectation (rad)
Minor damage state	-4.8529	β_{mi}=0.1085	0.0079
Major damage state	-4.1586	β_{ma} =0.1079	0.0157
Collapse damage state	-3.4426	β_{co} =0.2579	0.0331

3. SEISMIC RESPONSE ANALYSIS

In order to determine the fragility curve of the transmission tower, it's necessary to determine seismic responses of the tower induced by different magnitude of earthquakes. Due to the randomness of the ground motion, the seismic performance of the building will respond with uncertainty as well. The randomness of the ground motion is realized by building a package of various ground motions covering a wide range of peak intensity, time-varying amplitude, strong-motion duration and frequency content.

In this study, the maximum rotation angle of the top is taken as the seismic response parameter and peak ground acceleration (PGA) taken as seismic intensity parameter. A package of seismic records covering different conditions is chosen for the ground inputs of nonlinear dynamic analysis. By using regression analysis of the results, the relationship between the seismic intensity parameter and seismic response parameter of the structure is established.

The ground motion records are downloaded from the website of PEER (Pacific Earthquake Engineering Research), each of them has different site conditions, PGA, spectrum characteristics and duration to represent the variation of nature as much as possible. The maximum of PGA in all records is 1.779G and the minimum one is 0.109G. One record in which PGA= 0.753G , is taken for example, as illustrated in Fig. 8. The modal damping ratio is assumed to be 1%, and the seismic input is applied only in X direction. Corresponding to the seismic input shown in Fig. 8, the displacement response of the tower at the top is plotted in Fig. 9.



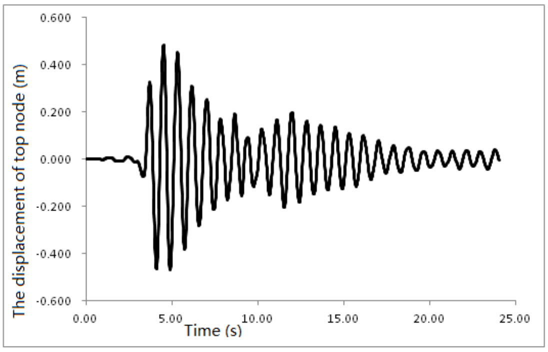
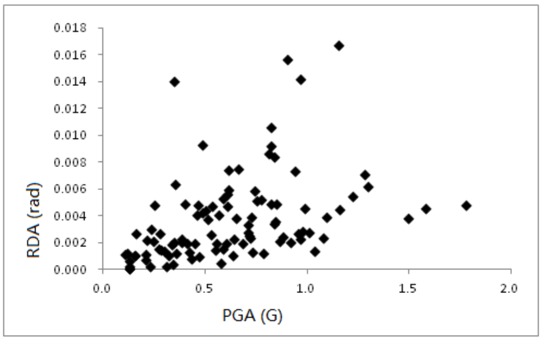


Figure 8. Time history of ground acceleration

Figure 9. Displacement time history of top node

The maximum rotation angle of the top corresponding to the PGA of each seismic wave can be obtained. And the maximum RDA of the transmission tower is plotted against its corresponding PGA, as shown in Fig. 10. Transforming the coordinates into logarithm scale, the relation between RDA and PGA is demonstrated in Fig.11.



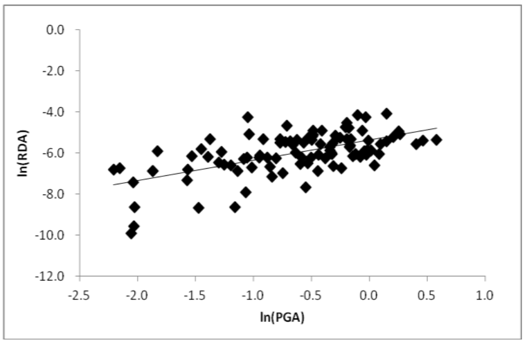


Figure 10. Seismic intensity vs. Seismic performance

Figure 11. Logarithmic description

The linear correlation between PGA and RDA can be determined by regression analysis, as shown in Eq. (2).

$$\ln(RDA) = 0.993\ln(PGA) - 5.354 \tag{2}$$

The statistical histogram of RDA in Fig.12 shows that the seismic response parameter has a lognormal distribution, which is coincident with the original assumption. Two-parameter lognormal distribution functions are used to represent the probability model of response parameters. By estimation method we could obtain the statistical parameters of the seismic performance, the mean value of its logarithm is -5.5994 and the standard deviation of its logarithm $\beta = 1.0555$.

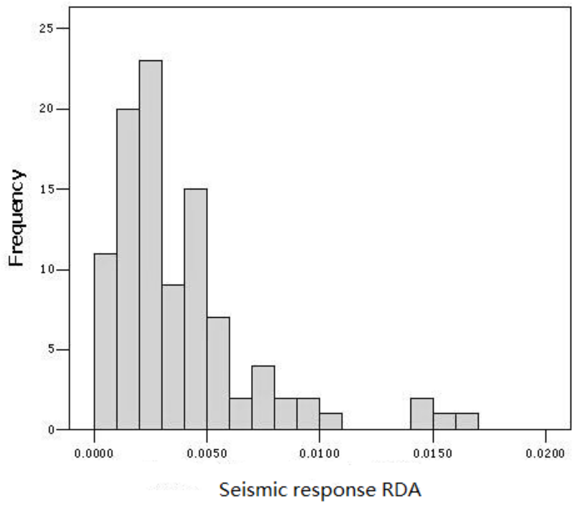


Figure 12. Histogram of seismic performance (rad)

4. FRAGILITY ANALYSIS

The fragility describes the probability of structural failure or damage states under a certain seismic intensity; it depends on the structural integrity, damage conditions and other factors. Commonly, the fragility curve with respect to seismic intensity is assumed to have a lognormal distribution characterized by two parameters. The probability that structural seismic response S_d exceeds the structural capacity R_c (R_c is the displacement limit of each damage state which has been determined in the previous performance analysis) can be calculated by the Eq. (3).

$$P_f = P_f(\frac{R_c}{S_d} \leq 1) \tag{3}$$

Both R_c and S_d obey a logarithm normal distribution. Therefore, probability of collapse or damage states can be determined by the Eq. (4).

$$P_f = \Phi\left[\frac{-\ln(\frac{\tilde{R}_c}{S_d})}{\sqrt{\beta_c^2 + \beta_d^2}}\right] \tag{4}$$

where $\tilde{R}_c$ is the mean value of R_c, S_d is structural response (seismic demand), β_c is logarithm standard deviation of seismic capacity, β_d is logarithm standard deviation of structural response, and $\Phi(\bullet)$ is a function of standard normal distribution.

According to the previous results which are shown in Fig. 10 and 11, the probability of minor damage, major damage and collapse state can be derived by substituting the corresponding data, the seismic fragility curve of the tower can be depicted in Fig.13. Even under the extremely intensive earthquake whose PGA is equal to 1.0G, the probability of tower collapse is less than 5%, the probability of major damage happened to the transmission tower is less than 15% and that of minor damage is less than 30%. It is obvious that the seismic capacity of transmission tower is very robust and the tower is not easy to collapse under seismic load.

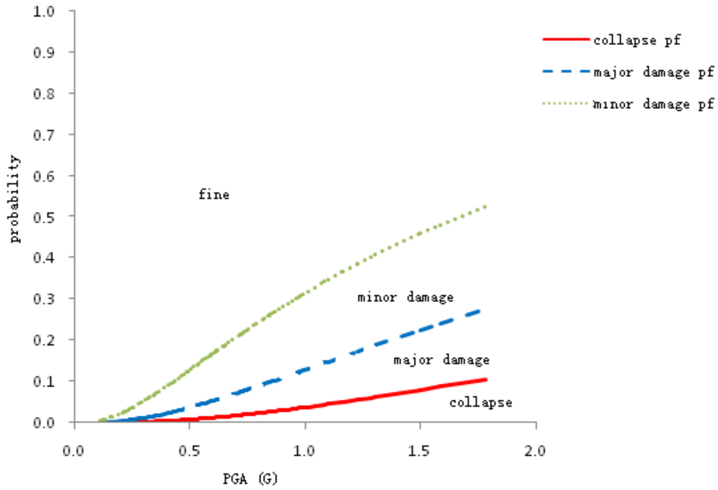


Figure 13. The seismic fragility curve of the transmission tower

5. CONCLUSIONS AND DISCUSSIONS

The paper presents a numerical method to obtain a seismic fragility curve of the transmission tower, which is very important to evaluation of the integrity and reliability of transmission towers. Considering the internal uncertainty of the tower, the randomness of ground motion and the variation of its seismic performance, seismic performance is analysed by using nonlinear buckling analysis method and nonlinear dynamic analysis. And the performance limits of different damage states are determined. Finally, the seismic fragility curve of the transmission tower is acquired by numerical Monte Carlo simulation. By the seismic fragility curve, the failure probability of the transmission tower under different magnitudes of earthquake can be visually predicted. However, there are several issues should be studied further in the future:

1. The characteristics of earthquake motion have three elements: time-varying amplitude, strong-motion duration, and frequency content. In this paper, the amplitude of ground motion (PGA) is the only factor describing the seismic intensity, while the relation between the other two characteristics and structural seismic response is left untouched.

2. The transmission tower itself is studied in this paper, neglecting the coupling effect of the conductor lines with the tower. In the long-span transmission-line system, the integrity and fragility of tower-line system needs to be studied further.

3. The damage state of the transmission tower is defined by the static nonlinear buckling analysis; however, the seismic load is dynamic having more complicate stability and safety conditions, for example, the dynamic instability. The damage state should be examined by dynamic analysis.

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