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Development of BP-based seismic behavior optimization of RC and steel frame structures

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ABSTRACT

The present study develops and numerically verifies a new methodology for the seismic behavior enhancement of reinforced concrete (RC) and steel frames incorporating BP (Back Propagation) algorithm and performance-based estimation. The proposed optimization flow allows automatic correction of the assumed damage weighting coefficients of the components with a series of prepared local and global damage indices defined as required. On the basis of the revised coefficients, modification of the sectional dimensions and corresponding reinforcement ratio of a six-story RC frame is carried out. The exceeding probability of each limit state and the Mean Annual Frequency (MAF) are reduced, leading to a better safety margin of the RC structure. And the mechanical model of a user defined element of self-centering energy dissipation brace (SCEDB) is experimentally confirmed. Meanwhile, the installation strategy against the normal continuous arrangement of the SCEDBs in a nine-story benchmark steel frame is proposed as well. As a result, the story drift ratio and local damage of the steel frame are effectively decreased. The BP-based optimization results demonstrate that the seismic performance of the two structures has been improved without any cost increase, resulting in an effective structural optimization method.

1. Introduction

Earthquakes are highly random and hard to be forecasted, posing a serious threat to human life safety and have a wide impact on human normal production activities. Previous earthquakes showed that structural deterioration and collapse had the greatest responsibility for economic losses. Therefore, in order to ensure better seismic behaviors of structural system on the premise of no obvious augment in construction costs, it proves to be necessary to seek the weak members and failure rules for the improvements of the corresponding adverse failure modes. Different optimization measures have different computational efficiency and optimum effects. Therefore, the optimization of structural failure mode has become a hot issue in the seismic design and research field.

A good optimization flow in industry requires optimized objective, optimization algorithm and improved techniques. As a result, the performance-based design of structures is a topic of growing interest, especially incorporating structural optimization methods. Stratis et al. [1] proposed a truss optimization method using the contrast-based fruit fly optimization algorithm and confirmed its good and robust performance. However, the algorithm should be improved in the case of high-dimensional search and complex domain spaces. Zacharenki et al. [2] presented an algorithm for the reliability-based seismic design of structures incorporating approximate performance estimation methods. Vamvatsikos [3] proposed an optimal multi-criteria seismic design method of highway bridges with the help of the static pushover. Both the two researches above are based on the SPO2IDA method, so the accuracy of the optimization results may be affected by the relevant errors. Papadopoulos et al. [4] proposed a structural optimal design method based on random vulnerability which employed structural damage level as the constraint criterion. Bruno et al. [5] proposed a new design methodology to evaluate the optimum configuration of network arch bridge schemes. Georgios et al. [6] developed a structural optimization framework for seismic design of multi-story composite buildings, aiming at minimizing the total cost of materials without structural seismic performance degradation. Beck et al. [7] considered structural and seismic randomness, and enforced a multi-objective optimization of a 3-story steel frame. Most of the previous work regards the stochastic algorithm as the 'engine' to search the local optimal solutions, and they are confined to a narrow parameter domain. The convergence of the algorithm is closely related to the computation time.

Recently, Genetic algorithm (GA) [8] and iterative algorithm have

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been widely concerned in structural design and optimization, which stands out against the problems of crude estimations of the building capacity and conservative design. GA comes from the thoughts of neural network which contains the crossover and mutation of gene sequences, absolutely belonging to a sort of stochastic algorithm. And the BP algorithm [9] learns from the error between the true value and predicted value. By propagating the error back, the weighting coefficients ω_{ij} is modified for several times, and the corrected coefficients directly show the importance of each cell and can be used to define the affinities between the branches and the output terminal. Compared with GA optimization, the BP algorithm certainly takes less computation time owing to deterministic optimization with the modified ω_{ij} instead of numerous numerical operations of random samples. It possessed a self-learning ability to extract the reasonable solution rules automatically. Therefore, this study has tried to introduce such a classic algorithm into the field of structural design and optimization, and finally proved to be a feasible measure.

These previous optimization methods seem to have one thing in common that strengthening the weak parts of the structure found by given measures can effectively improve the seismic performance. In contrast, the earthquake-sensitive components are defined as objects that are supposed to be strengthened or protected in this paper. Some researches [10–13] described the relationship between components damage and whole structural damage by using the combination coefficients, which in most cases are identical, failed to distinguish the sensibility among these components and structures. The BP algorithm is employed to modify the combination coefficients in order to implement the optimization procedure under the guidance of the weighting coefficients. Two case studies are carried out to verify the effectiveness and the feasibility of the BP-based optimization method. Modification of sectional dimensions and steel ratio is used for the RC frames, and the comparisons of the exceeding probability of each limit state and the Mean Annual Frequency (MAF) is conducted. Otherwise, the benchmark steel frames is reinforced with SCEDBs [14–15] whose mechanical model is corrected and verified by several groups of experimental results, moreover, the installation locations of SCEDBs depend on the results of the BP algorithm. Comparative analysis of the seismic performance of the frames with different SCEDBs installation strategies is carried out as well.

2. Evaluation of seismic behavior

It is required to determinate the principle of damage index calculation, aiming to accurately quantify the damage degree of structure under earthquakes. Structural damage can be classified into three levels, material damage, components damage and whole structural damage. Material damage can fundamentally describe the micro process of performance degradation. Damage and failure of components show the weak positions and failure paths from the macroscopic response, however, single component damage may not lead to the collapse of the entire structure. As the study of integration from components damage to whole structural damage is inadequate, the ductility damage index *DI*, proposed by Powell and Allahabadi [16], is used in this paper. It can distinctly describe the damage development and performance deterioration of the whole structure during the strong ground motion with a simple form, and is given by,

$$DI = \frac{\delta - \delta_y}{\delta_u - \delta_y} = \frac{\mu - 1}{\mu_u - 1} \tag{1}$$

where δ is the maximum displacement of structure during the earthquake, δ_y is the yielding displacement and δ_u is the ultimate displacement, $\mu = \delta/\delta_y$ is the ductility demand, $\mu_u = \delta_u/\delta_y$ is the ductility capacity. These two eigen-displacements can be easily obtained by a pushover analysis, where δ_y is equal to the displacement when the initial stiffness of the structure comes to an obvious decline, δ_u is equal to the displacement when the bearing capacity drops to 80% of the peak capacity in the pushover curve. The loading mechanism of pushover analysis is proportional to the first vibration mode. *DI* is always equal to 0 when $\delta < \delta_y$, considering the structure in an elastic and damage-free condition.The seismic design code [17] ensures that the structure is in a normal working status by limiting the maximum inter-story drift ratio, $\max(\theta_{drift})$. In this study, when the maximum drift ratio is approaching or exceeding the elastic-plastic limit, the frame is considered to lose bearing capacity. Meanwhile, the distribution of drift ratio along the height represents the failure mode of the structure. The elastic and elastic-plastic limit of $\max(\theta_{drift})$ are respectively 1/550 and 1/50 for RC frames in China. According to the standard above, the first limit state LS_1 and the second limit state LS_2 are defined as follows,

$$LS_1\text{: } \max(\theta_{drift}) \geqslant 1/550 LS_2\text{: } \max(\theta_{drift}) \geqslant 1/50 \tag{2}$$

In this study, the structural seismic behavior is evaluated from the perspective of damage and probability. As incremental dynamic analysis (IDA) [18–20] is one of the most powerful seismic performance estimation methods and can eliminate different types of uncertainties, the entire design process selects MAF to assess the structural seismic capacity. The MAF can be given by the convolution integral of the limit-state fragility curve with the site hazard curve. Each limit-state fragility curve can be obtained by regression analysis of the IDA results while the site hazard curve is presented by seismic hazard analysis. Thus, according to the total probability theorem, the MAF of each limit state is given by,

$$\nu(LS_i) = \int_0^{+\infty} P(LS_i|IM = im)\left|\frac{d\nu(IM)}{dIM}\right| dIM \tag{3}$$

where $P(LS_i|IM = im)$ is the exceeding probability of LS_i, the *i*th limit state, when the intensity measure *IM* is equal to *im* which is the peak ground motion (PGA) in this paper, $|d\nu(IM)/dIM|$ is the absolute value of the slope of the site hazard curve. Assuming that $\max(\theta_{drift})$ obeys a lognormal distribution, the contingent probability $P(LS_i|IM = im)$ can be given by,

$$P(LS_i|IM = im) = \Phi\left(\ln\frac{\widehat{\max(\theta_{drift})}}{\max(\theta_{drift})_{lim}}/\sigma\right) \tag{4}$$

where $\ln(\widehat{\max(\theta_{drift})})$ and σ are respectively the maximum likelihood estimation of mean value and standard deviation of $\max(\theta_{dirft})$, $\Phi()$ is the cumulative probability function of the standard normal distribution.

The key to identifying earthquake-sensitive components by BP algorithm is to use the difference between the actual value (whole structural damage) and the predicted value (component damage integral). Note that the accuracy of structural local damage is the key to a precise BP algorithm. For RC members, an improved Park-Ang [21] damage index that defined as the linear combination of the maximum displacement and the dissipated energy is given as follows,

$$d_{ij} = (1-\beta_0)\frac{\delta_{m,ij}}{\delta_{u,ij}} + \frac{\beta_0 \int dE}{f_y(\delta_{u,ij}-\delta_{y,ij})} \tag{5}$$

where d_{ij} is the damage index of the *i*th RC column at the *j*th story, β_0 is the combination coefficient that takes 0.1[22] in this study for simplicity, $\delta_{m,ij}$ is the relative maximum displacement of the *i*th RC column at the *j*th story during the ground motion, $\int dE$ is the corresponding hysteretic energy, $\delta_{u,ij}$ and $\delta_{y,ij}$ are respectively the ultimate and yielding displacement of the *i*th RC column at the *j*th story.

For the steel frames, based on Eq. (1), the section damage DI_s is given by,

$$DI_s = \max(DI_{s,a}, DI_{s,b}) \tag{6}$$

where $DI_{s,a}$ and $DI_{s,b}$ are respectively given by,