



Research Article

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Damage analysis on bridge structures of typical rail transit

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ABSTRACT

Seismic damage was the main reason for damage and collapse of structure. One of the development directions of performance-based aseismatic design was to quantize seismic damage and evaluated seismic resistance by damage analysis. This paper expounds the Park-Ang criterion, which is widely used for damage analysis, and its insufficient is pointed out. The modified Park-Ang two-parameter damage evaluation model is used in this paper, two kind of methods are adopted to define damage index. By analysis the damage conditions of typical rail transit bridges under various strength level in Beijing area, it is proved the result of two analysis methods have a good consistency. This paper offers a good reference for aseismatic performance evaluation and design of rail transit bridges.

Key words: Rail transit bridges; ground motion intensity; damage index; aseismatic performance

INTRODUCTION

Seismic damage is the main reason for damage and collapse of structure. One of the development directions of performance-based aseismatic design is to quantize seismic damage and evaluate seismic resistance by damage analysis. To be able to identify whether the control member parts has been damaged according to the result of nonlinear dynamic response analysis under seismic ground motion input, over the years, researchers make every efforts to build a criteria, which can distinguish whether a member has been damaged under random inelastic displacement loop, according to the results of tests on different reinforced concrete members in which the members are repeated loaded until being damaged.

In recent years, based on the present research results, scholars tried to research the material and structure damage using the relating theories and technologies. In the field of civil engineering, researchers tried to research concrete material and structure by adopting these new research results, while, to find a scientific damage assessment method is always one of the focuses. Lubliner et.al [1] presented the fracture energy-based plastic damage model, in which the difference between tension and compression performance of concrete is considered and the ratio of the current area to total area (fracture energy) in the stress-strain curve is used to define damage parameter. Based on Lubliner's model, in consideration of damage conditions such as breaking, crushing and rigidity degeneration under cyclic loading, Lee et.al [2] adopted two damage parameters to describe the tension and compression damage behavior. Barbat [3] proposed an overall damage index based on potential energy, and this model was used in structure seismic damage evaluation. Hanganu et.al [4] introduced the finite element method that calculates the overall damage index based on energy equation, this method can be used to predict both partial and overall damage of structure system, and calculate its ultimate bearing capacity and failure load. Tikhomirov [5] put forward plastic coupling damage model of reinforced concrete structures, the anisotropic damage in cement matrix and plasticity of the interface between matrix and aggregate were used to describe the nonlinear constitutive behavior of concrete. According to Continuum damage mechanics, fracture mechanics and the concept of plastic hinge, Marante [6-7] proposed the general formula for calculation of cumulative damage of the reinforced concrete frame. Modern seismic design allows the corresponding control part of the reinforced concrete structures enter into the state of inelastic deformation under intensive earthquake loading. Earthquake excitation in the main quake and aftershock may cause amplitude with random deformation cycle, for normal reinforced concrete structures, the Cycles can range from

several to dozens of times, and much of the loop will make corresponding control component parts entering yield deformation state. Tests show that the damage conditions of reinforced concrete members under this loading are different from the process under static load. It depends on the material cumulative damage degree under loading, including the damage accumulation of concrete and steel bar itself, and the damage accumulation of the effect of bonding and crack interface effect. It is generally believed that the damage cumulative of material itself and the effects of bonding and crack interface effect, including the damage accumulation after entering the state of inelastic, is achieved by two ways: One is when the stress or strain (including sliding) exceeds a certain threshold, the damage will be increased with the force enlarged. if the stress or strain continues to increase, eventually, damage will happen when the damage cumulative surpasses the corresponding degree. Another is in the process of cyclic loading, the stress or strain caused by the damage accumulation is still not enough to cause failure, but damage accumulation will increase with the increase of cycling times, sustained growth of cycles can also lead to the extent of the damage. Therefore, structure damage accumulation in random inelastic deformation cycle caused by earthquake can be regarded as the integrated embodiment of two damage accumulation process.

Structure seismic performance generally includes elastic and inelastic performance (damage performance of structure). The aim of structural seismic design is to prevent structure from damage, especially structure damage under the action of earthquake. Therefore, commonly the seismic performance refers to the damage performance of structure. However, the current seismic design method neglected the low cycle fatigue cumulative damage of structure under intensive earthquake. The single largest deformation checking calculation is difficult to control the damage performance under seismic action. The modified Park-Ang two-parameter damage evaluation model is adopted in this paper. Two kinds of methods are adopted to define damage index. By analysis the damage conditions of typical rail transit bridges under various strength level in Beijing area, this paper offers a good reference for aseismic performance evaluation and design of rail transit bridges.

DAMAGE INDEX

Park-Ang damage performance evaluation model

Currently, the model used for the bridge structure seismic damage assessment is the cast reinforced concrete structure seismic damage model. It mainly divided into two categories: strength-based damage model and reaction-based damage model. These models require single or multiple damage parameters, such as seismic maximum reaction, plasticity and ductility and cumulative energy consumption, and these parameters are mainly determined by the nonlinear dynamic analysis. Of many damage criterions, the two-parameter damage criterion proposed by Y.J.Park and A.H.S.Ang [8] in 1985 and expressed by the member maximum deformation and cumulative hysteretic energy linear combination is the most widely used. Its form:

$$DI = \frac{x_m}{x_{cu}} + \beta \frac{E_{hs}}{F_y x_{cu}} \quad (1)$$

or

$$DI = \frac{\mu_m}{\mu_{cu}} + \beta \frac{E_{hs}}{F_y x_y \mu_{cu}} \quad (2)$$

$$\beta = (-0.447 + 0.073\lambda + 0.24n_0 + 0.314\rho_l)0.7^{100\rho_w} \quad (3)$$

where: F_y , x_y respectively, are yield shear and yield displacement of components ; x_{cu} , μ_{cu} respectively ,are ultimate displacement and the ultimate displacement ductility factor of components ; x_m , μ_m respectively, are the maximum reaction displacement and maximum response displacement ductility factor of components ; E_{hs} , β are cumulative hysteretic energy and energy dissipation factor of members ; ρ_l is reinforcement ratio of entire longitudinal steel ; ρ_w is reinforcement ratio by volume ; λ is the shear span ratio ; n_0 is axis.

The reason why Park-Ang criterion is widely used and thereafter some improved forms are based on it, appears to be due to the following two advantages of the criterion:

- (1) The guidelines could have used that time to collect all kinds of component test results do calibration: initially took 261 test results in the subsequent empirical coefficient as important for major revision number of the reference test to 402;
- (2) The guidelines by the displacement of a combination of items and energy items that conceptually and destruction by large deformation amplitude (damage accumulation first approach) and repetitive loading effects (damage

accumulation second way) caused by the combined effect of the basic rules match.

The modified Park-Ang damage evaluation model

Although the Park-Ang model is widely used, while, Park-Ang model also has obvious theoretical insufficient: (1)in the stage of elastic deformation, when $E_H=0$, DIPA should be 0, while, according to formula (1), $DIPA>0$;

(2) under the condition of single-direction loading, when the peak deformation was reached, DIPA should be 1, it indicates that the structure is in ultimate state of collapse, while by formula (1), $DIPA>1$.

In consideration of these insufficient of Park-Ang model, based on responsibility and practicability of engineering, this paper adopts the two-parameter Park-Ang damage evaluation model modified by Jiang hui, Zhu xi et.al[9], the following modified model, which can resolve these questions effectively, is defined:

$$DI = \frac{\delta - \delta_y}{\delta_u - \delta_y} + \beta \frac{E_H}{F_y(\delta_u - \delta_y)} \quad (4)$$

$$DI = (\mu - 1)/(\mu_u - 1) + \beta E_H / F_y \delta_y (\mu_u - 1) \quad (5)$$

Formula (5) are the pattern that shows formula (4) by deformation ductility, in formula (5)the definition of β 、 δ 、 δ_u 、 E_H 、 F_y are the same as formula (4), δ_y is yield deformation, μ 、 μ_u are practical deformation ductility and the ultimate deformation capacity under monotonic loading. For the confirmation of ultimate deformation capacity under monotonic loading μ_u ,the new Zealand specification requires that [10], the structure should be afford to bear four cycles with horizontal displacement of $\mu_u u_y$,in the same time, the decrease of bearing capacity can not exceeds 20%. In this paper, to make comparison easier, in all calculation, the ultimate deformation ductility under monotonic loading are supposed to be constant.

To define performance index specifically, firstly, the relationship between damage index and damage scale showed in the following table, which is provided by Hindi and Sexsmith (2001), is used to define specific measurement value of structure under different expected damage performance. It is given according to the RC structure performance distribution under cycling seismic loading.

Table 1 The relationship between damage index and damage provided by Hindi and Sexsmith

Damage index	Damage level
$DI \leq 0.1$	No damage
$0.1 \leq DI < 0.2$	Slight injury, minor cracking and easy to maintenance
$0.2 \leq DI < 0.4$	Moderate damage, severe cracking, protective layer spalling, can repair
$0.4 \leq DI < 0.6$	Serious injury, a lot of cracking, steel exposed, repair difficulties
$0.6 \leq DI < 1.0$	Serious injury, crushing of concrete, steel bar buckling, beyond repair
$DI \geq 1.0^*$	Serious injury, crushing of concrete, steel bar buckling, beyond repair Completely collapsed

In addition, FEMA 273 divided the performance into “Operational (OP)”、“Immediate Occupancy (IO)”、“Life Safety (LS)” and “Collapse Prevention (CP)”four levels. According to decrease value of structure stiffness under each performance level absorbed from engineering experiences and Hybrid structure shaking table test,and by distinguishing the slope variation of IDA curve with this standard, Bu yi, Lv xilin et,al proposed the range of each interval, it is showed in table 2.

Table2 The corresponding performance level of each performance points and slope drop of IDA curve

Performance level	Operational	Immediate Occupancy	Life Safety	Collapse Prevention
Performance point	OP	IO	LS	CP
slope drop of IDA curve(10%)	10	20	50	80

ENGINEERING CASE

outline of bridge

For the continuous bridge of yudai River Street in Beijing subway, the main span distribution is 25m+32m + 25 m, and the width of the bridge is 8.85m. The concrete of C50 is used for the main beam, while the bridge piers prefer the concrete of C30. The size of the triangular section for bridge pier is 2m*1.6m, and the diameter of the circular section for bridge pier is 1.8m, with the diameter of the pile foundation is 1m. See the Fig.1 for detail size of the bridge section.

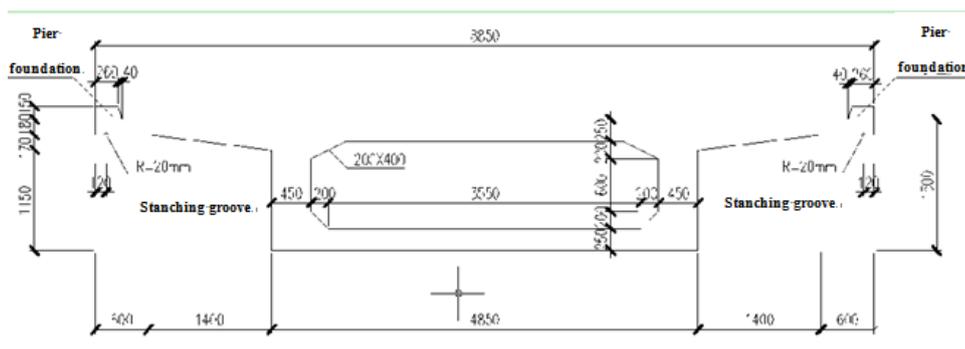


Fig. 1 Size of the section for main box beam

Reinforcement of piers are showed in Table 3.

Table 3 Reinforcement of pier cross section

No.	Longitudinal ratio of reinforcement(%)	The volume ratio of stirrup reinforcement(%)
4# and 7#	1.27	0.28
5# and 6#	1.16	0.36

The choice for the record of surface movement

In this paper, the record used to calculate DI is a wave of California Landers earthquake in 1992. The earthquake is Mw 7.3 earthquake happened in southern California Landers (Landers), the epicenter is located in the southeast of the mojave desert, about 180 km east of Los Angeles, destructive earthquake is very big, the surface rupture length of more than 70 square kilometers.

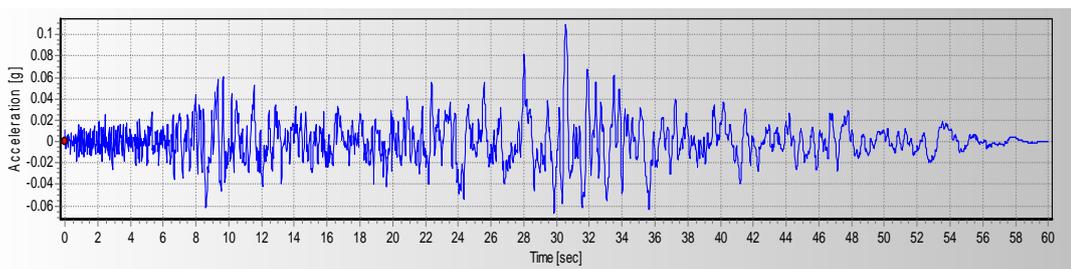


Figure 2 the original record of Landers earthquake

Near the epicenter, Surface dislocation is about 3 m, and in the north of the fault, the dislocation is 6.7 m , after the Flanders earthquake, as many as tens of thousands of aftershocks happened during the following months. The selected seismic record belongs to the typical earthquake records, the moment magnitude 7.3, epicentral distance is 55.7 km, the site predominant period is 0.5 s, PGA is 0.5. as input of seismic records, it is adjusted by spectral acceleration Sa (T1).

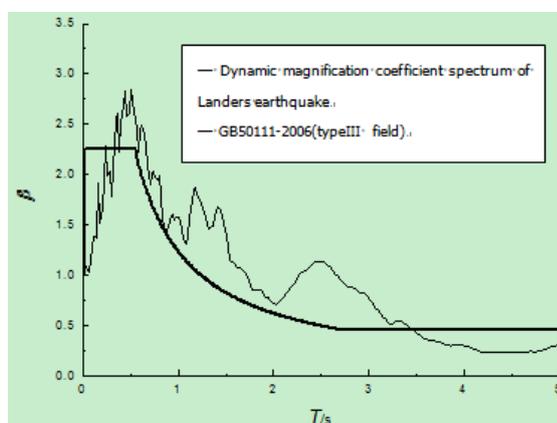


Figure 3 the contrast between dynamic magnification coefficient spectrum of seismic record the specification line

From figure 3, the selected dynamic magnification coefficient of seismic record spectrum has better accordance with the specification line. Over the long-periodic, intensity is larger than standard spectral line, explains the selected seismic records belongs to strong earthquake records in soft field.

THE BRIDGE DI CURVE OF URBAN RAIL TRANSIT

The results of single pier

Analyze the Yudai river street continuous beam bridge on the Beijing subway Batong line,since the middle piers 5# and 6# suffered greater binding effect,the damage was also more serious under the seismic effect.as shown in Diagram 4a) and b):

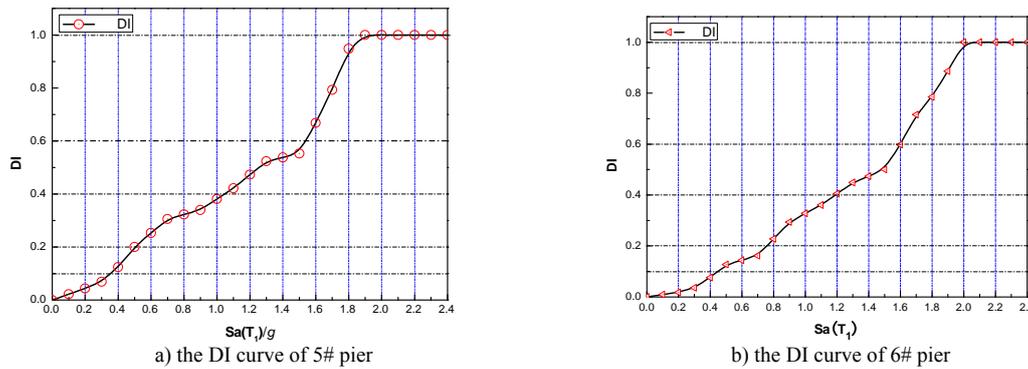


Diagram 4 The DI curve of single pier

The calculated results of full-bridge

By weighting the each pier DI value of main bridge of the Yudai River street continuous beam bridge according to DI. We get the continuous bridge DI curve of the whole typical urban rail transit.as shown in Diagram 5.

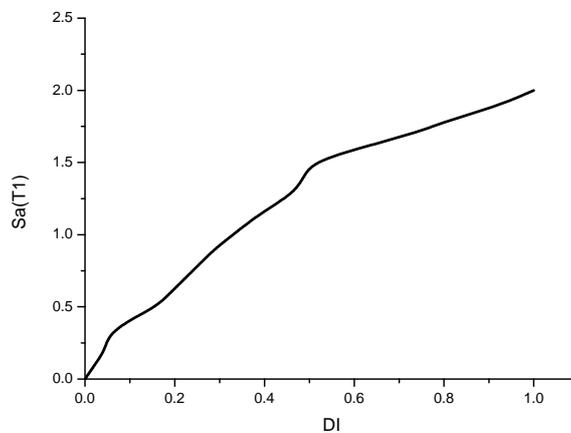


Diagram 5 The DI curve of full-bridge

From the DI curve shown in Diagram 5 it can be seen , for the $DI \leq 0.1$ stage without damage , the corresponding Sa (T1) of the bridge is from 0.0g to 0.4g and the corresponding PGA is from 0.0g to 0.197g; for the $0.1 \leq DI < 0.2$ slight damage stage, the corresponding Sa (T1) of the bridge is from 0.4g to 0.6g and the corresponding PGA is from 0.197g to 0.295g; for the $0.2 \leq DI < 0.40$ moderate damage stage , the corresponding Sa (T1) of the bridge is from 0.6g to 1.15g and the corresponding PGA is from 0.295g to 0.566g; for the $0.4 \leq DI < 0.6$ serious damage stage, the corresponding Sa (T1) of the bridge is from 1.15g to 1.58g together with the the corresponding PGA from 0.566g to 0.778g;The bridge reached the collapsed limit at 1.96g with the corresponding PGA of 0.965g.

The DI value of different property point can be given according to the decline of the slope of the IDA curve shown in Table 2.as shown in Table 4.

Table 4 The DI value of each property point

property point	OP	IO	LS	CP
DI	0.087	0.13	0.47	0.92

Through Table 1 and Table 3 it can be seen , the DI value of the performance point OP is 0.087 , means the structure is in stage without damage ; the DI value of the performance point IO is 0.13 , then the structure is in stage with slight damage ; the DI value of the performance point LS is 0.47 , then the structure is in stage with serious damage difficult to repair; the DI value of the performance point CP is 0.92 , then the structure is in stage with severely damaging beyond repair, and it's not much differ from the DI value (1.0) of the collapse limits.

CONCLUSION

The article first elaborated the Park-Ang guideline which was widely used for damage assessment and pointed out its deficiencies. On this basis it used the modified Park-Ang two-parameter damage assessment model as the damage performance evaluation model. The model can solve the shortcoming $DIPA > 1$ in the elastic reaction stage reached the maximum deformation when $EH=0$, $DIPA > 0$ and the one-way loading condition. The continuous beam bridge model of the typical urban rail transit was established and select a wave of the 1992 Landers earthquake in California for damage assessment, then given the damage curves of the single pier and full-bridge, then selected the damage index and destruction hierarchical relationship given by Hindi and Sexsmith (2001) in Table which were used to define the structure specific metrics under different expected damage performance, then given the DI value of different performance points according to the decline of the curve slope. Through analysis and comparison it can be found that: the structural states analyzed by two methods were in high consistency, one more the urban rail transit continuous bridge with good ductility and under strong earthquake effect with a good seismic performance.

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