Seismic Vulnerability Analysis of Cable-Stayed Bridges with Steel-Concrete Composite Beams Considering Erosion by Wind and Sand

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Abstract. Due to erosion by wind and sand, the steel beam coating of cable-stayed bridge becomes thin or even falls off, resulting in corrosion of steel beam. In addition, the strength of concrete tower columns also decreases due to the erosion area of sand. In order to study the damage of the main bridge, cable and column of the steel-concrete composite cable-stayed bridge with consolidated pier and tower beams during the whole life cycle, the ground peak acceleration was used as the seismic input strength index, and the damage and failure probability of the steel-concrete composite cable-stayed bridge were studied based on IDA vulnerability analysis method. The results show that: From the perspective of the vulnerability analysis of the whole bridge, the damage probability of all structures under the action of earthquake is basically the same within 20 years after the initial construction of the bridge. When rare earthquakes come, all structures basically fail at the same time. However, after 20 years, the damage probability of the structure is main beam > cable tower > cable tower. The safety reserve of the cable is higher.

Keywords: steel-concrete composite girder cable-stayed bridge; erosion by wind and sand; seismic vulnerability analysis

1 INTRODUCTION

As an important part of lifeline engineering, with the rapid development of highways in China, cable-stayed Bridges are widely used due to the rugged terrain in northwest China and the advantages that they can cross steep canyons. However, the harsh environment in Northwest China, such as the erosion of bridge structures by wind and sand, leads to damage to steel beam coating and concrete [1-2]. Therefore, many scholars began to study the influence of wind-blown sand on concrete. Zhang

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Yushuang [3] found that erosion cracks and spalling of cement blocks would occur in concrete structures under the action of sand erosion. Pan Xiaotian [4] obtained through test and simulation that the concrete strength after erosion would be reduced to some extent. However, all parts of the bridge would have been damaged to varying degrees under the earthquake. Jion [5], under the excitation of seismic waves from 20 Class III sites, concluded that the vulnerable areas of inverted "Y" shaped towers of cable-stayed Bridges often appear in three parts: the bottom of the tower, the bottom of the middle tower and the top of the middle tower. zhong [6] used numerical simulation to deduce the limit state of the section of the pylon and its failure location; Ma Kai [7] studied the influence of spatial variability of ground motion on the vulnerability of long-span cable-stayed Bridges by using cloud map method and Monte Carlo simulation, and found that coherence effect and site effect had a greater impact on the vulnerability analysis. Then it is necessary to study the seismic performance of the structure under the influence of wind and sand erosion, and it is of great significance to evaluate the safety performance of the bridge affected by wind and sand erosion.

Therefore, this paper takes steel-concrete composite girder cable-stayed bridge as the research object, considers the influence of wind erosion and earthquake on the time-varying vulnerability of the structure, adopts IDA analysis method to analyze the time-varying vulnerability curve, and explores the variation rule of its vulnerability, which provides reference for practical engineering and performance-based seismic concept.

2 ESTABLISHMENT OF AEOLIAN SAND MATERIAL DEGRADATION MODEL

Sand erosion will reduce the thickness of the coating and the thickness of the concrete protective layer and weaken the stiffness of the structure. Therefore, 20 years as a time interval, divided into 6 working conditions, such as 0 years, 20 years, 40 years, 60 years, 80 years and 100 years as time nodes, calculate the corrosion of steel beams at each node and the reduction of concrete.

2.1 Cable Wind Erosion Analysis

According to a large number of investigations on the cables of existing cable Bridges and a large number of literatures [8], it can be seen that the average service life of cable sheaths under environmental influence is =12 years. According to the empirical formula in literature [9], steel wire corrosion rate parameters can be obtained, and the life of cable galvanizing layer can be calculated: =0.006/0.0011=5.45 years. Based on the 20-year cable replacement design, it can be seen that the remaining life is =20- (+) =2.55 years. Combined with the reference [10], the broken wire rate is 1.45%, the remaining thickness of the steel wire after 20 years of construction can be calculated as 2.86505mm.

2.2 Analysis of Steel Beam Wind Erosion

According to the life prediction formula of composite coating subjected to sand erosion in literature [10] and the exponential model calculation formula of steel beam corrosion depth, the corrosion depth, reduced area and corrosion rate of steel beam affected by sand erosion in different years were obtained, as shown in Table 1. Values of A and B were obtained according to literature [11].

Table 1. Calculation table of corrosion depth, reduced area and corrosion rate of steel beam caused by sand erosion in different years

Time	20 years	40 years	60 years	80 years	100 years
corrosion penetration（mm）	2.92432	5.84863	8.77295	11.69727	14.62158
Reduction of area（m^2）	0.10703	0.21406	0.32109	0.42812	0.53515
corrosion rate（%）	5.10	10.18	15.26	20.35	25.44

2.3 Analysis of Tower Wind Erosion

Finnie[12] proposed a formula for calculating the amount of erosion wear by studying the erosion wear test. The wind speed at the location of the bridge was 20.7m/s. Combined with literature [13], the reduction of the area of concrete eroded by wind-blown sand was calculated as shown in Table 2.

Table 2. Relation between erosion quality of wind-blown sand and C50 concrete area and time varying age

Time	20 years	40 years	60 years	80 years	100 years
weight eroded（kg)	4.09×10^{-5}	8.17×10^{-5}	1.23×10^{-4}	1.63×10^{-4}	2.04×10^{-4}
Reduction of area(m^2)	2.86×10^{-2}	5.72×10^{-2}	8.58×10^{-2}	1.14×10^{-1}	1.43×10^{-1}

3 FINITE ELEMENT MODEL ESTABLISHMENT

The example is a highway bridge across the Yellow River. The main span of the bridge is 155+296+155=606m. The width of the bridge is 36.6m. The main beam adopts double I-beam composite beam. The cable is made of Φ7mm high-strength galvanized parallel steel wire, the standard strength is 1770MPa, and the cable is protected by double-layer PE sheath. The main tower is reinforced concrete H-shaped bridge tower, and the main tower is made of C50 concrete. The height of tower No. 16 and No. 17 is 113m.

The full bridge model of cable-stayed bridge is established based on finite element software. The main beam and tower are built with beam elements, the cable is built with truss elements, and the main beam is a composite steel-concrete beam, so the joint section simulation is adopted. Due to sand erosion and rust, the area of the material changes without affecting the inherent characteristics of the material, such as elastic modulus. Therefore, in the finite element calculation, the area change is taken as the parameter to conduct seismic response and vulnerability analysis for the change, as shown in Figure 1.

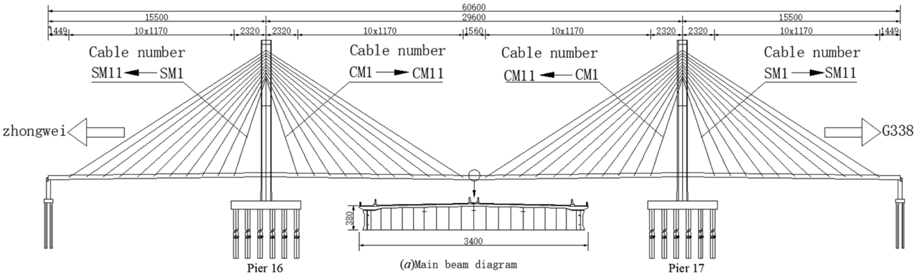


Fig. 1. Full bridge layout

4 SEISMIC VULNERABILITY ANALYSIS

4.1 Earthquake Vulnerability Calculation Formula

The seismic vulnerability of a structure is defined as the conditional probability that a structure reaches or exceeds a certain limit state at a given level of seismic intensity: $F_{\phi}(X)$ Called seismic vulnerability function [14]

$$F_{\phi}(X) = P[T \geq R|IM = x] = \Phi[\frac{\ln m_{T|IM} - \ln m_R}{\sqrt{\eta_T^2 + \eta_R^2}}] \tag{1}$$

$$\ln m_{T|IM} = \lambda_0 + \lambda_1 \ln(IM = PGA) \tag{2}$$

The formula: $\Phi[\bullet]$ Is the distribution function under standard plus; IM Is the intensity parameter of ground motion; $T \geq R$ Indicates that the structure reaches or exceeds a certain limit state, T For seismic response, R For earthquake resistance m_T Is the median seismic response value of the structure; m_R Is the median seismic capacity of the structure; η_T Is the logarithmic standard deviation of seismic response; η_R Is the logarithmic standard deviation of structural seismic capacity; λ_0 and λ_1 It can be obtained by log-linear regression through the results of nonlinear time history analysis. Based on the experience points provided by HAZUS99, When IM takes ground peak acceleration and PGA as independent variable, $\sqrt{\eta_R^2 + \eta_T^2}$ Take 0.5[15].

4.2 Seismic Wave Selection

The peak acceleration of ground motion in the area where the bridge is located is 0.20g, the characteristic period of response spectrum is 0.45s, and the project site category is class II. 12 seismic waves conforming to the characteristics of the site were selected, and the standard and average response spectra were shown in Figure 2. Then, the PGA of 12 seismic waves is adjusted to 10 different intensity levels of 0.1g~1.0g, and 120 seismic waves are obtained.

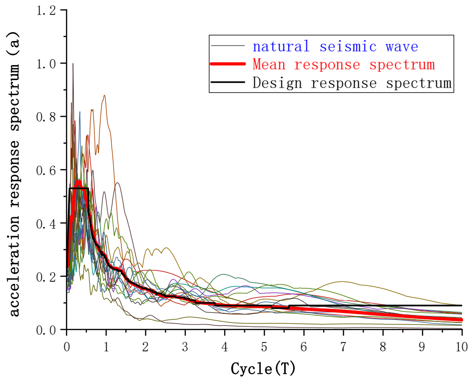


Fig. 2. Response spectra of 12 seismic waves (5% damping)

4.3 Definition Of Structural Damage Limit State

Definition of Cable Damage Limit State. Refer to reference [16] to define the limit damage state, $f_{y1} = 0.6R_y^b$ Is defined as a slight damage limit state, $f_{y1} = 0.75R_y^b$ Is defined as the medium damage limit state, Reaching 85% of the ultimate tensile strength is defined as the severe failure limit state, and reaching the nominal ultimate tensile strength is the complete failure limit state. The calculation of damage limit states at all levels is shown in Table 3.

Table 3. Damage limit states of cable at all levels

quantitative index	Slight failure	Moderate failure	Serious failure	Complete failure
Tensile stress (MPa)	$1062 < f \le 1327.5$	$1327.5 < f \le 1504.50$	$1504.5 < f \le 1770$	$1770 < f$

Definition of Damage Limit State of Cable Tower. The displacement of pier top and curvature of control section of the bridge with high pier and long span do not appear synchronously, and the relationship between material damage and deformation is not one-to-one correspondence. Therefore, the bending moment-curvature analysis of the control section is carried out, and the curvature is used as the basis for damage evaluation of pier column. Figure 3.Quantification is divided into four levels,for example $\phi \le \phi_y'$ For minor damage; $\phi_y' < \phi \le \phi_y$ Moderate failure; $\phi_y < \phi \le \phi_u$ For serious damage; $\phi_u < \phi$ For complete destruction, (explain: ϕ` Is the curvature of the longitudinal bar when it first yields, ϕ_y Is equivalent yield curvature, ϕ_u Is the curvature of the core concrete when crushed) 。Table 4 shows the maximum curvature of the lower section of the left pylon to the middle pylon of Pier No. 17 in different years.。

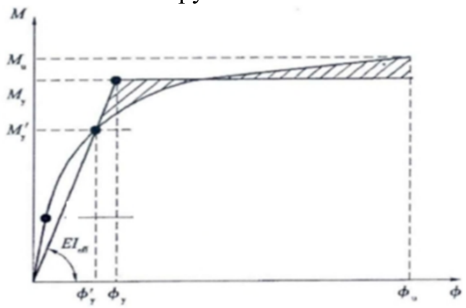


Fig. 3. moment-curvature curve of lower section of tower column

Table 4. Relation table of cross section curvature (x10^{-3}/m) at the lower end of the tower column in the left tower of Pier 17

analysis parameter	faulted condition	0 years	20 years	40 years	60 years	80 years	100 years
First yield	Slight failure	0.275434	0.275449	0.275465	0.275539	0.275675	0.275885
Equivalent yield	Moderate failure	0.686977	0.694083	0.699707	0.708726	0.718354	0.728139
$\varepsilon = 0.004$	Serious failure	0.924435	0.934364	0.941314	0.954869	0.967447	0.981302
ultimate curvature	Complete failure	1.574782	1.571404	1.566838	1.562622	1.561471	1.543113

Definition of Damage Limit State of Steel-Concrete Composite Beams. Refer to China's "JTG_D64-2015_ Highway Steel Structure bridge design Code" 4.2.3 and "Highway Bridge Technical Condition Evaluation Standard JTGTH21-2011" 8.3.3 to define the damage level of the bridge, take L/400 as the complete failure limit, L is the calculated span, the specific description refer to the above code. The failure state of the main beam is divided into four types, and the specific definitions are shown in Table 5.

Table 5. Relationship between deflection of main beam and damage state

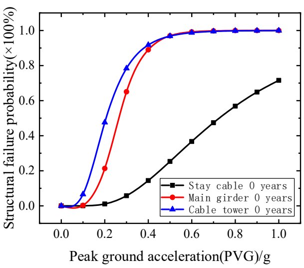
quantitative index	Slight failure	Moderate failure	Serious failure	Complete failure
Deflection(mm)	$\omega \leq 246.3$	$246.3 < \omega \leq 493.3$	$493.3 < \omega \leq 740$	$740 < \omega$

4.4 Construction of Seismic Vulnerability Curve of the Whole Bridge

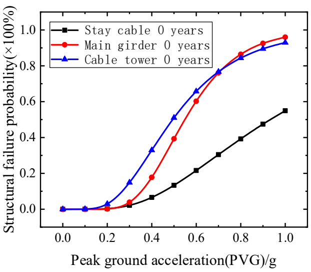
Taking 20 years as the node of the time system, the structure after sand erosion at each time point was modeled, and parameters corresponding to different times were input into the model. Then, the seismic responses of 120 cables, towers and main beams under seismic loads were characterized by logarithmic stress, deflection and curvature, and the two coefficients λ_0 and λ_1 of formula (2) were obtained by linear fitting, as shown in Table 6 below. Equation (1) was used to analyze the seismic vulnerability of each damage state, and the time-varying seismic vulnerability curve of the wind-sand erosion cable-stayed bridge was finally fitted, as shown in Figure 4 below.

Table 6. Linear fitting parameters of cable stress, tower curvature, main beam deflection and ground peak acceleration (PGA)

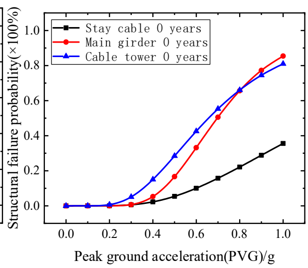
parameter		0 years	20 years	40 years	60 years	80 years	100 years
digging line	λ_0	0.8907	0.8756	0.8814	0.8795	0.8508	0.8378
	λ_1	1.6712	1.7632	1.7633	1.7635	1.7632	1.7631
cable bent tower	λ_0	1.0444	1.1697	1.1906	1.2115	1.2324	1.2533
	λ_1	-6.5471	-6.4162	-6.2852	-6.1543	-6.0233	-5.8924
kingpost	λ_0	1.4580	1.4067	1.3527	1.2955	1.2349	1.1703
	λ_1	2.1821	2.4003	2.6185	2.8367	3.0549	3.2732



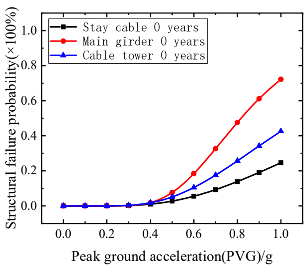
（a）0 years-Slight failure



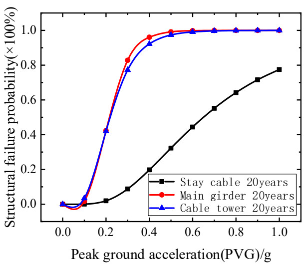
（b）0 years-Moderate failure



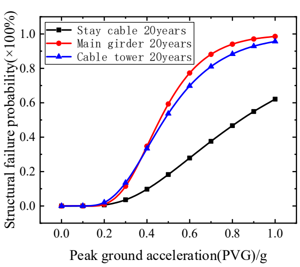
（c）0 years-Serious failure



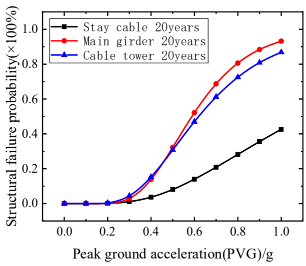
（d）0 years-Complete failure



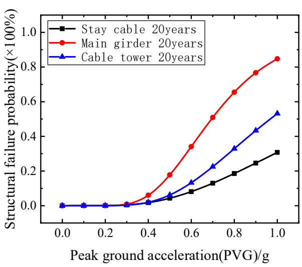
（a）20 years-Slight failure



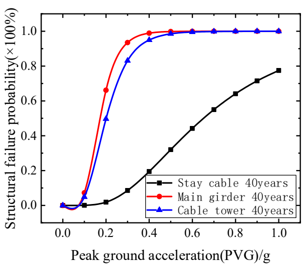
（b）20 years-Moderate failure



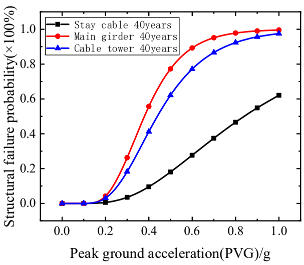
(c) 20 years-Serious failure



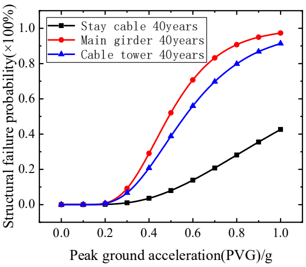
(d) 20 years-Complete failure



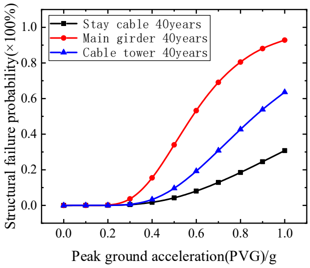
(a) 40 years-Slight failure



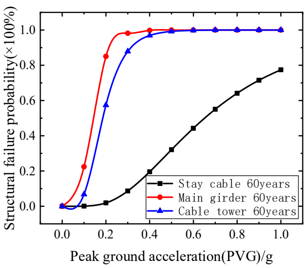
(b) 40 years-Moderate failure



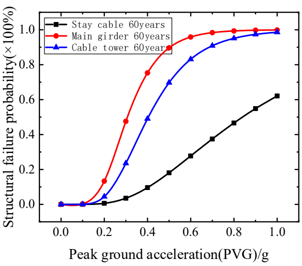
(c) 40 years-Serious failure



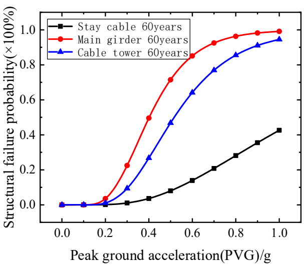
(d) 40 years-Complete failure



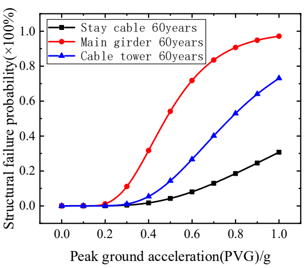
(a) 60 years-Slight failure



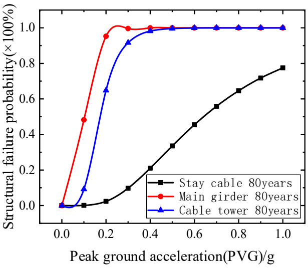
(b) 60 years-Moderate failure



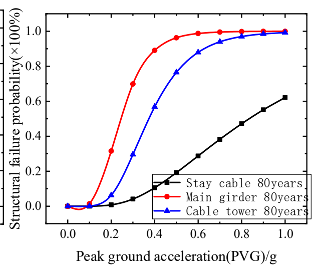
(c) 60 years-Serious failure



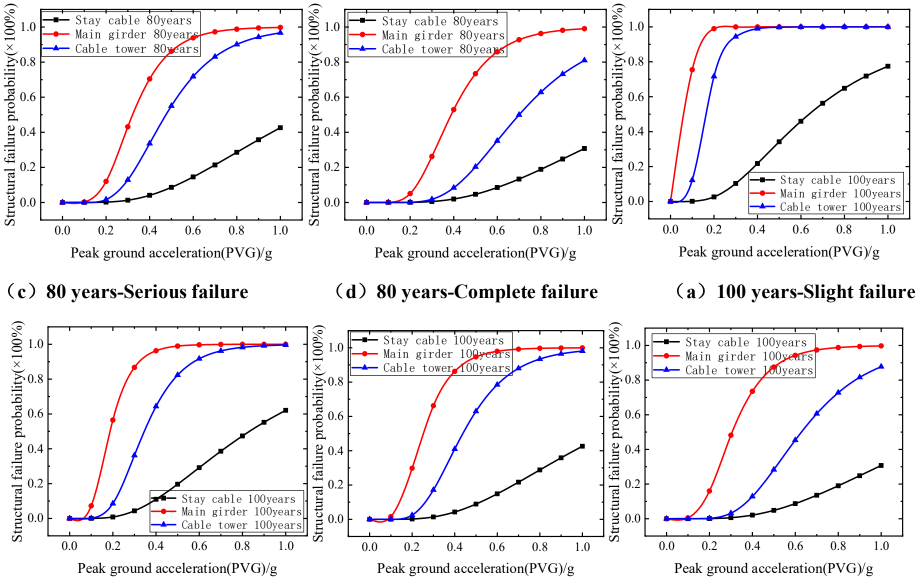
(d) 60 years-Complete failure



(a) 80 years-Slight failure



(b) 80 years-Moderate failure



(c) 80 years-Serious failure (d) 80 years-Complete failure (a) 100 years-Slight failure

(b) 100 years-Moderate failure (c) 100 years-Serious failure (d) 100 years-Complete failure

Fig. 4. Comparison of seismic vulnerability of all components of cable-stayed bridge

Affected by the erosion of time-varying wind and sand, the structure will have different degrees of damage, which will change the seismic performance of the structure. Since the cable is replaced once every 20 years according to the requirements of the design specification, the time interval is 20 years, so the failure probability of the cable remains basically unchanged during the time-varying process. However, the section area of the tower and main beam is reduced due to time-varying wind erosion and environmental corrosion, and the seismic performance is also reduced. As shown in FIG. 4, it is not difficult to find that the probability of minor damage, moderate damage, severe damage and complete damage to the main beam of the whole bridge under the time-varying effect is always higher than that of other structures after 20 years. At 40 years, when the probability of serious damage of the main beam exceeds 50%, the corresponding PGA=0.5g, the probability of serious damage of the main beam, cable tower and cable is 52.05%, 38.8% and 7.96%, respectively. At 60 years, when the probability of serious damage of the main beam exceeds 50%, the corresponding PGA=0.4g, and the probability of serious damage of the main beam, cable tower and cable is 50%, 26.73% and 3.61%, respectively. In 1980, when the probability of serious damage of the main beam exceeds 50%, the corresponding PGA=0.4g, and the probability of serious damage of the main beam, cable tower and cable is 70.42%, 33.60% and 4.05%, respectively. When the probability of serious damage of the main beam exceeds 50% in 100 years, the corresponding PGA=0.3g, and the probability of serious damage of the main beam, cable tower and cable is 66.20%, 17.12% and 1.34%, respectively. This shows that time variation has a great influence on the main beam. In

100 years, when the failure probability of the main beam exceeds 50%, only when the PGA is 0.3g, the serious damage probability reaches 66.20%.

5 CONCLUSIONS

Through the analysis of the steel-concrete composite girder cable-stayed bridge under the action of sand erosion, the following conclusions are obtained:

(1) The comparative analysis of the vulnerability of the whole bridge shows that each structure is greatly affected by the erosion of wind and sand, and its failure probability increases with the change of time; With the increase of earthquake intensity, the failure probability of each structure increases significantly, showing the same law. Therefore, it is necessary to consider the influence of sand erosion on the seismic performance of Bridges during service.

(2) Repair maintenance was not considered in the vulnerability analysis of main beams and piers, and the purpose of the study was to analyze the seismic vulnerability of main components of cable-stayed Bridges under natural erosion. The research results also showed that, in order to ensure the safety of Bridges, regular testing and timely maintenance of Bridges must be carried out during the bridge life cycle, and designers should consider the effect of wind erosion when designing.

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