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Seismic Performance of Cable-Stayed Bridges Subjected to Different Types of Ground Motions

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Summary

Seismic response of the cable-stayed (CS) bridge under the earthquake are not consistent with the regular bridges for their different dynamic characteristics, in particular, when the CS bridges are forced to the near-fault ground motions with velocity pulse. However, most of current literatures tended to study the seismic behavior of the CS bridges by using the deterministic analysis approach, and these literatures not compared the variability of seismic response for the bridge subjected to different ground motions. Seismic fragility function is readily appropriate as an probabilistic analysis tool to quantify the performance of bridge due to its advantage to account for the uncertainty associated with ground motions. A representative medium-span concrete cable-stayed bridge is taken as an example, this study will firstly compare the seismic response of the bridge subjected to both near-fault and far-fault ground motions with identical peak ground accelerations (PGA), in order to highlight the obvious distinction. Moreover, two earthquake bins (far-fault bin and near-fault bin) are assembled to account for the uncertainty derived from ground motions. Because the common used PGA was proved to be inapplicable for fragility analysis of the long-period structures. Therefore, four common used intensity measures, e.g. PGA, PGV, PGD and SA, are selected to identify their priority for the fragility analysis of the CS bridge. Finally, the seismic fragility curves of some important components with respect to the optimal intensity measures are developed for the case-study bridges subject to two different ground motion bins.

Keywords: Earthquake, Cable-Stayed Bridge, Ground Motions, Fragility.

1. Introduction

Medium-span concrete cable-stayed (MSCCS) bridges are a focus of interest for bridge engineers worldwide due to their fantastic appearance, economic advantages and the improvement of construction technique. Currently, the number of MSCCS bridges is far more than that of the long-span steel cable-stayed bridges. And these bridges usually play a vital role in highway and railway transportation networks. However, the weighty concrete superstructure of MSCCS bridges makes them much more vulnerable to unpredictable earthquakes compared to the lightweight steel cable-stayed bridges, which have been verified by the damage investigation of Chi-Lu Bridge in the 1999 Taiwan Chi-Chi earthquake. Therefore, it is an urgent job to investigate the seismic performance of MSCCS bridges.

Structural response to far-fault and near-fault ground motions are quite different for both of the signal degree freedom (SDF) system [1] and multiple dimensional structures, such as moment-resisting frame structures and bridges [2]. Influences of the near-fault ground motions on structural response are much stronger than that of the far-fault ones, especially when high energy velocity pulses are included at the beginning time of the near-fault history records [2, 3]. In addition, the spectral accelerations of near-fault ground motion are much greater than that of ordinary ones during the range of long period in spite of their comparable PGA values [2]. However, the fundamental periods of the MSCCS bridges are longer than that of the regular bridges. This unique characteristic will make them more sensitive to the forward-directivity effect [4] and the long period district of the spectrum curve [5], which used to be ignored in previous investigation for short-to-medium period bridges. Actually, how this coherent long period pulses affect the dynamic response of cable-stayed bridge is still questionable. The influence of different ground motions on responses of MSCCS bridges can be analysed by using the nonlinear time history analysis that incorporates various earthquake records.

In addition to conventional deterministic approach, the probabilistic approach has an important advantage to account for various uncertainties. In the probabilistic approach, seismic fragility evaluation is identified as one of the key elements together with hazard analysis and loss analysis. Currently, seismic fragility function has been used widely to quantify the seismic performance of bridges in the form of fragility curves [6-9]. Though many algorithms have been proposed to develop the fragility curves in past decade [10-12], the theoretical fragility curves based on dynamic analysis are popular with engineers and researchers to assess the seismic performance of bridges in regions that lack earthquake damage data [13]. In addition, some researchers investigated the seismic fragility of various bridge types in regards to the classification of seismic regions. These studies tended to focus on regular girder bridges that account for majority of as-built bridges, but as a result, the conclusions cannot be applied directly to MSCCS bridges. Recently, some studies focused on the seismic fragility of cable-stayed bridges [14, 15], but they cannot consider the effects of near-fault ground motions. Therefore, it is a big challenge to study the probabilistic seismic performance of MSCCS bridges subjected to different ground motions.

In this study, the deterministic analysis is first applied to investigate and highlight the influence of long-period velocity pulse on the investigated MSCCS bridges subjected to three different ground motions. Uncertainties of ground motions are considered by assembling two ground motion bins that include or exclude velocity pulse. The seismic fragility curves for some weighted components are developed to compare the probabilistic seismic performance of the MSCCS bridges.

2. Bridge Description and Modelling

Figure 1 illustrates the configuration of the investigated MSCCS bridge and the modeling details of various seismic components. This superstructure of bridge is composed of three spans with length of 132+264+132m, which is supported by two transition piers and 64 pairs of steel cables that anchored to two diamond shape concrete towers (see Fig. 1(a)). SAP2000 software (Computers and Structures Inc. 2012) is used to build the finite element (FE) model of the cable-stayed bridge. This study adopts the large-displacement truss elements with a modified elasticity modulus to model the nonlinear characteristic of supporting cables. To simplify the analysis process, this study assumes the superstructure and upper concrete towers to maintain elasticity under the earthquake, but considers the nonlinearities of the lower concrete towers and transition piers. The fast nonlinear analysis (FNA) algorithm embedded in SAP2000 software is applied to improve the computational efficiency. To accommodate with the algorithm of FNA, the pivot plastic elements are used to simulate the nonlinearities of these concrete components. Because the pile foundations are embed into the bedrock and the surrounding soil is very stiff, the soil-structure interaction (SSI) effects are ignored in this paper. However, other significant components, such as the longitudinal restrained cable, the pot sliding bearing, transverse retainer and expansion joint, are modeled by using the nonlinear link elements in SAP2000 software. It should be noted that the damage of supporting cables are not considered in this study, because they are supposed to keep elastic under huge earthquake.

The simplified bilinear curve is applied to the pivot model. To calculate the model parameters of different concrete sections, the authors programed the moment-curve (M-Φ) analysis procedure with MATLAB software. In this procedure, the stress-strain relationships of undefined and defined concrete fibres are modeled according to the research of Mander et al. [16], and the behaviour of steel fibre is modeled by using a bilinear model with the strain-hardening ratio equal to 0.01.

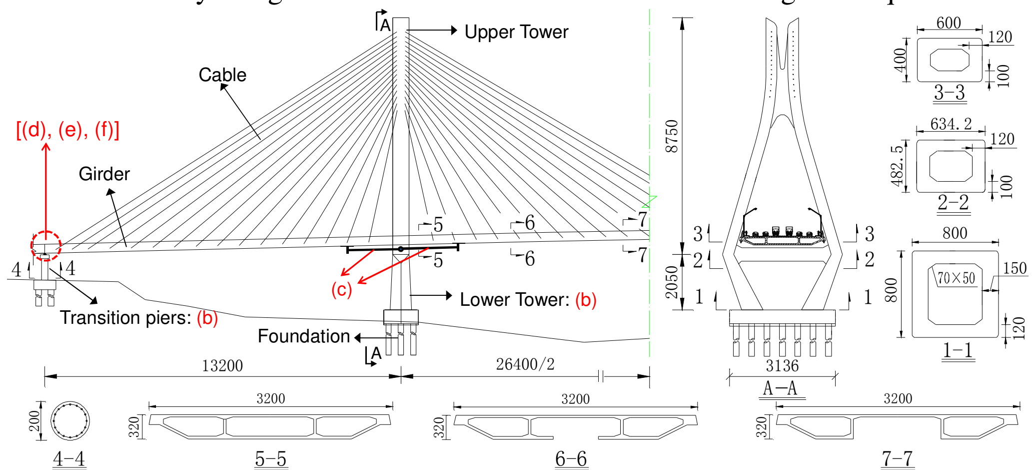

Fig.1 Configuration of the case-study bridge

Take the section 1-1 as an example, Figure 2 plots the computational M-Φ curve and the idealized moment-curvature curves. The nonlinear behaviour of the bilinear is described in the SAP2000 software by the moment-rotation relationship (M-Φ). Therefore, if a constant curvature over the length of the plastic hinge (L_p) is assumed, the following equations can calculate the rotation [17]

$$\theta = \phi L_p \quad (1)$$

$$L_p = 0.08L + 0.022 f_{ye} d \geq 0.044 f_{ye} d \quad (2)$$

where L is the length from the inflection point of the flexure to the section with the maximum moment, and f_{ye} and d are the yield strength and the diameter of the longitudinal reinforcing bars, respectively. Although, these formulas are not rigorous to calculate the plastic hinge length of the tapered tower. However, it is still difficult not find another more appropriate formula from previous literatures. To simply the FE modeling process, we assumed that these formulas were applicable for the concrete towers in this study. Then, the parameters of pivot models for the investigated concrete sections in two directions are shown in Table 1.

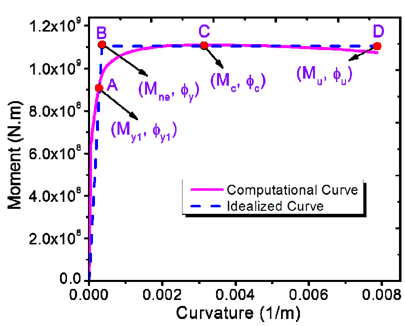

Fig.2 Comparison of the M-Φ analysis

Table 1. Parameters of Pivot Models

Sec.	Dir.	M_{ne} $MN \cdot m$	Φ_y $10^{-4} rad$	M_u $MN \cdot m$	Φ_u $10^{-3} rad$
1-1	L	1124.6	3.01	1254.3	14.81
	T	1195.4	2.82	1256.2	17.61
2-2	L	749.00	2.89	759.00	15.21
	T	570.20	2.47	620.21	12.42
3-3	L	655.80	3.21	705.77	13.38
	T	437.40	2.87	497.36	12.62
4-4	L	14.930	5.11	15.610	28.38
	T	14.930	5.11	15.610	28.38

Sec. =section; Dir. =direction; L=longitudinal; T=transverse

3. Deterministic Analysis

Three distinguished ground motion records are selected to conduct the nonlinear time history analysis of the case-study bridge. These records belong to the near-fault record with pulse-like (NF-P), near-fault without pulse-like (NF-NP) and far-fault without pulse-like (FF-NP) ground motions, respectively. Figure 3 shows the comparison of the basic characteristics for the three selected ground motion records, including the response spectra and the time history curves of accelerations and velocities. One should note that, the peak ground velocities (PGVs) of various records are absolutely distinguished each other even though their peak ground accelerations (PGAs) are almost identical. The PGV of near-fault record is larger than that of the far-fault ones. Especially, the PGV value of the N-P record is up to 111.9cm/s, which far larger than the other two PGVs. In additions, the illustration of response spectra shows that, in the range of short period (T<0.1s), the spectral accelerations of the three records are comparable each other. In the range of medium period (0.1s<T<1.5s), the spectral accelerations of records without pulse-like (N-NP, F-NP) are bigger than that of the one with pulse-like (N-P). However, in the range of long period (T>1.5s), the spectral accelerations of N-P tend to be far bigger than that of the other two records. As is well known, the cable-stayed bridges are long-period structures whose fundamental period is far longer than 1.5s. Such as the case-study bridge, the first period is up to 3.72s in the longitudinal direction. Therefore, the seismic response of cable-stayed bridge subjected to N-P records is supposed be more intensity than that subjected to ordinary ones.

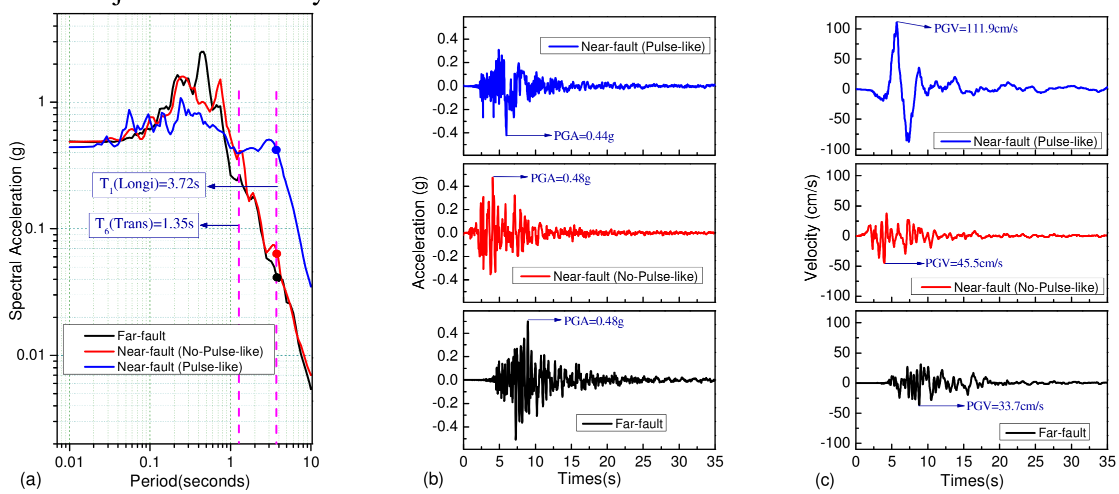

Fig.3 Characteristics of three records: (a) response spectra; (b)acceleration; (c) velocity

Figure 4 and Figure 5 present the comparisons of longitudinal and transversal responses for some key components of the case-study bridge subjected to three selected records, including the time-history response of the girder displacements, the hysteresis curve of moment-rotation for the bottom section of concrete tower and the hysteresis loops of force-deformation for the transverse retainer on the transition piers.

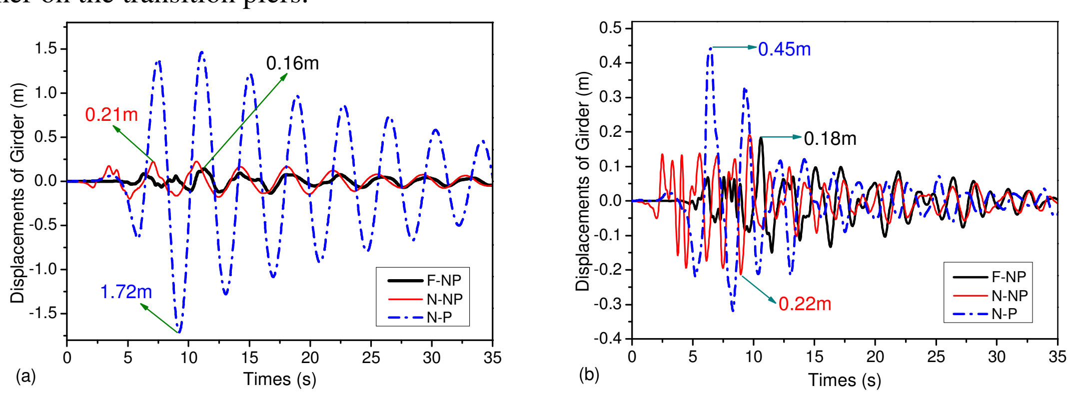

Fig.4 Comparison of time history response of girder displacement under three records in the (a) longitudinal and (b) transverse directions

Figure 4 shows that whether in the longitudinal or transverse directions, the displacement response of the girder caused by velocity pulse is much larger than that caused by other two records. In particular, in the longitudinal direction, the difference is so obvious that the maximum displacement of girder under N-P record is up to 1.72m, which is approximately 8 to 11 times more than that of no pulse-like cases. Figure 4 (b) shows that the maximum displacement in transverse direction is still caused by the N-P record, the value is equal to 0.45m that is only a quarter of the maximum value in the longitudinal direction. However, for the no pulse-like records, the transversal displacements (0.22m and 0.18m) are similar to longitudinal ones (0.21m and 0.16m) under the same ground motions.

Figure 5 (a) shows that the bottom section of concrete tower is forced into plasticity by the excitation of the N-P record. However, it is still able to keep elasticity when the bridge is subjected to two records without velocity pulse-like. One may note that, the hysteresis loops of moment-curvature for the tower section in the transverse direction are not illustrated in this paper, because no ground motion can excite the nonlinear behaviour. In other words, the tower section can always keep elasticity in the transverse direction under any of the three selected ground motions. Therefore, only elastic deformations are occurred to the concrete tower in the transversal direction, which could be one of the reasons for smaller transverse displacement of the girders. On the contrary, the larger longitudinal displacements of the girder are partially due to the plastic deformation of concrete tower.

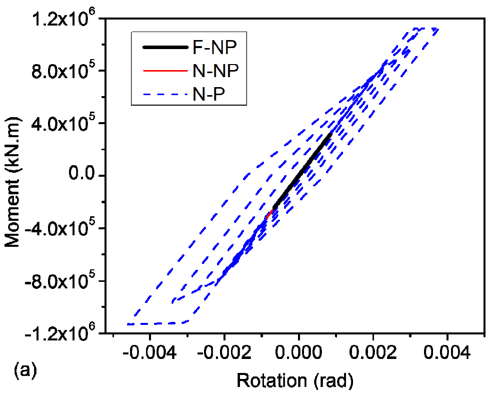

Fig.5 Seismic response comparison for the moment-rotation of the tower

The deterministic comparison of structural responses demonstrated that the dynamic characteristics of various ground motions produce severe effects on the nonlinear time-history analysis results. PGA is not a good parameter to measure the intensity of ground motions when both the far-fault and near-fault ground motions are considered, but PGV values of three selected records are in accordance with the comparison results. However, the deterministic analyses cannot account for the possible variability derived from selection of ground motions and the modeling process. Therefore, it may produce contradicting results when different ground motions are used to conduct time-history analysis. In next section, the probabilistic analysis will provide a clear and more confidence analyses result to testify the previous deterministic comparisons.

4. Probabilistic Analysis

4.1 Ground motions

The deterministic analyses indicates that the velocity pulse-like is the primary case to affect the seismic response of cable-stayed bridges. Therefore, in this section, two ground motion bins, consisting of 100 pairs of near-fault records (pulse-like) and 100 pairs of far-fault records (no pulse-like), were selected for investigation. These ground motions are mainly based on the work of Baker et al. [18] and the PEER earthquake database (http://ngawest2.berkeley.edu/). The case-study bridge is a typical long-period structural form with complex nonlinearity. Therefore, it is necessary to use a wide range of IMs to determine their priorities for fragility analysis. In this study, four

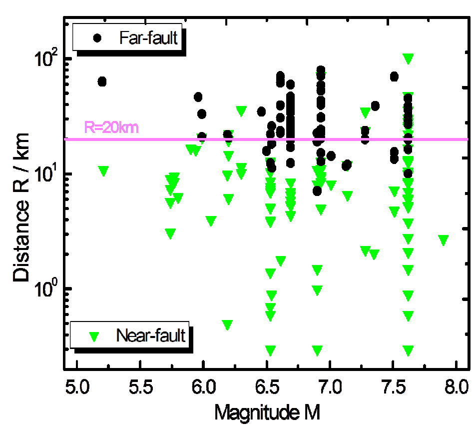

Fig.6 Distribution of ground motions

typical intensity measures: peak ground acceleration (PGA), peak ground velocity (PGV), peak ground displacement (PGD) and the spectral acceleration (SA) are adopted for investigation and comparison, respectively. Hereinto, the spectral acceleration (SA) is defined as the spectral acceleration at the geometric mean of the fundamental period of the longitudinal and transverse directions with the 5% damping ratio.

4.2 Seismic fragility functions

Seismic fragility function describes the conditional probability of bridge's failure as a function of the ground motion intensity. There are many different methods to obtain the seismic fragility functions of a particular bridge, but analytical fragility was widely applied in regions lacking structural damage or earthquake data [13]. In fragility analysis, both the seismic demand and the capacity of the bridge were assumed to follow the lognormal distributions. Therefore, the fragility function can be expressed as follows[7]:

$$P_f = \Phi\left[\frac{\ln(IM) - \ln(\mathrm{IM_m})}{\zeta_f}\right] \qquad (3)$$

where $\Phi[\cdot]$ is the standard normal distribution function; $IM_m = \exp((\ln(Sc) - \ln(\mathrm{a}))/b)$ is the median value of the intensity measures for the specified limit states and $\zeta_f = \sqrt{\beta_{EDP|IM}^2 + \beta_c^2 + \beta_M^2} \big/ b$ is the corresponding modified dispersion; $\beta_{EDP|IM}$ is the logarithmic standard deviation of the demand when the seismic intensity equals IM; and β_c is the logarithmic standard deviation of the structural capacity. β_M reflects the contribution of modeling uncertainty and $\beta_M = 0.03$ is acceptable and advisable for structural fragility analysis [19].

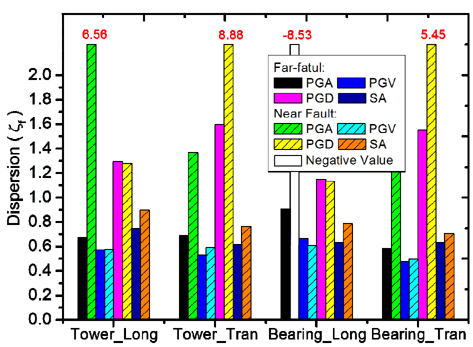

Fig.7 Comparison of dispersions to various intensity measures

According to the research of Padgett et al. [7], the modified dispersion ζ_f is able to reflect the influence of uncertainty derived from ground motions. Figure 7 shows the comparison of dispersions (ζ_f) for different earthquake demand parameters (EDPs) respect to various intensity measures. It is observed that the peak ground acceleration (PGA) performs well for all EDPs when subjected to the far-fault ground motions. However, the dispersion values of PGA are too big for the near-fault ground motions and it could be negative for some EDP. From Fig.7, we also can find that the dispersion of all EDPs respect to the peak ground displacement (PGD) is too big to predict the precise seismic demands. Contrarily, the other two intensity measures, both the PGV and the SA, are very suitable for the seismic fragility analysis, whether or not the velocity pulse-like is included in the ground motions. Therefore, this study will use the PGV and SA to develop the seismic fragility curves.

Figure 8 show the fragility curves of the tower section and bearing respect to the PGV under the excitation of different ground motions. It is observed that the bearing is more fragile than the concrete towers at four different damage states, which is actually in accordance with the seismic design criterion for the cable-stayed bridge. Because the concrete towers are much more important than the bearings to maintain the service function of the cable-stayed bridge. In addition, it can be seen from Figure 8 that, in the longitudinal direction, the failure probabilities of two components subjected to the near-fault ground motions are higher than that subjected to the far-fault ones. On the contrary, in the transversal direction, we can find the opposite conclusion, which the far-fault ground motions can result in more serious damage to the bridge. The comparison of the SA-based fragility functions is similar to that of the PGV-based fragility curves. Therefore, this paper did not illustrate the figure of SA-based fragility curves. From the figure 3, it is observed that the fundamental period of the case-study bridge is 3.72s in the longitudinal direction, and the first-order transversal period is only 1.35s. In this study, the spectral accelerations of far-fault ground motions is bigger than that of the near-fault ones in the range of medium period (0.1s<T<1.5s). However, the near-fault effect mainly occurs in the range of long period (T>1.5s). This phenomenon just explained the comparison results of fragility curves in Fig.8. In other words, the seismic fragility function of the bridge is actually dominated by both the structural dynamic characteristic and the selection of ground motions. In particularly, the effect of far-fault and near-fault ground motions might be different in two horizontal directions if the fundamental periods of the bridge are very distinguished in this two directions.

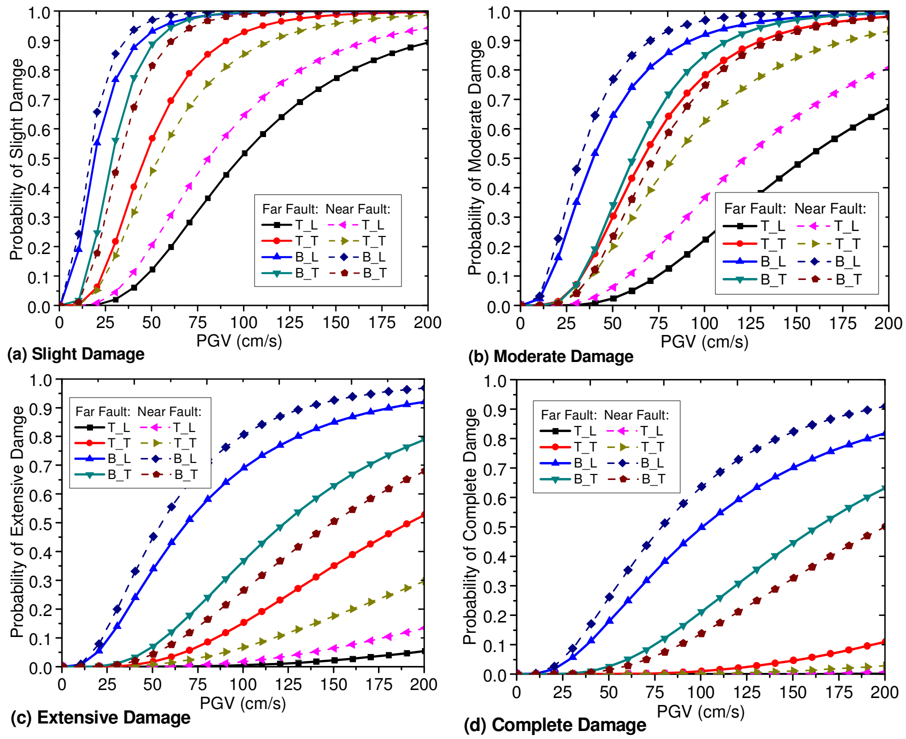

Fig.8 Comparison of seismic fragility curves of various components with respect to PGV for both the far-fault and the near-fault ground motions(T_L=Tower_ longitudinal; T_T=Tower_ transversal; B_L= Bearing_longitudinal; B_T= Bearing_transversal)

Conclusions

This paper investigates the influence of different ground motions on seismic behaviour of the median-span concrete cable-stayed (MSCCS) bridge by using both the deterministic and probabilistic analysis method. It is concluded that the velocity pulse of the ground motion will lead to more severe damage to the bridge. However, this near-fault effect mainly happens for the long-period structure, and it might result in inverted result in the range of short-median period. It should be noted that the above conclusions are obtained by adopting the peak ground velocity (PGV) or spectral acceleration (SA) as the intensity measure. Because the common used peak ground acceleration (PGA) is not suitable for the near-fault ground motions.

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