# EVALUATION OF THE SEISMIC PERFORMANCE OF REINFORCED CONCRETE BRIDGE COLUMNS BASED ON PLASTIC HINGE MODEL

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**ABSTRACT :**

Plastic hinge model has been widely used in bridge seismic design codes such as Japan, Caltrans, New Zealand and China (revised edition), to evaluate deformation capacity of RC bridge columns. With the development of bridge performance/displacement based seismic design, several damage indices have been suggested, such as ultimate curvature and curvature ductility factor of critical section, maximum strain of confined concrete and reinforced steels, low cycle fatigue damage indices of longitudinal reinforcement etc. To study the accuracy degree of damage indices calculated with plastic hinge models and the main influencing factor, a computer program was developed employing 5 plastic hinge models to compute aforesaid damage indices compared with test data of RC bridge columns. The study results show that force-displacement curves and residual deformation calculated match the experiment with adequate accuracy, but the strain of longitudinal steel is overestimated and the strain of core concrete is underestimated. The computed ultimate curvature is lower than experiment results when shear span ratio is not less than 8. It is also recognized that under the same loading control displacement, loading path hardly affects the aforesaid damage indices.

**KEYWORDS:** RC bridge pier, performance based seismic design, plastic hinge model, damage index

## 1. INTRODUCTION

Plastic hinge is permitted to form to dissipate energy at the bottom of RC bridge piers by ductility design method under attack of large earthquakes. Plastic hinge model assume flexural curvature distributes with idealization form along bridge piers, then elastic and plastic displacement on the top can be figured out, as shown in Fig. 1. Plastic hinge models have been adopted by bridge seismic design codes such as New Zealand (ZNS), Caltrans, Japanese (JSCE) and Chinese (JTJ004-2005) etc. The usage of plastic hinge model in Caltrans, Japanese and Chinese code is to calculate displacement (ductility) ability according to ultimate compression strain of concrete, while in New Zealand code curvature ductility is obtained according to displacement ductility with the model to design hoop reinforcement, which is related with curvature ductility.
Priestley and Park completed the initial study of plastic hinge model by a series of quasi-static test of RC bridge piers. They proposed plastic curvature be rectangular in plastic hinge zone and a statistic effective plastic hinge length is given, which is adopted by a number of bridge seismic design codes. Chang and Mander suggested a

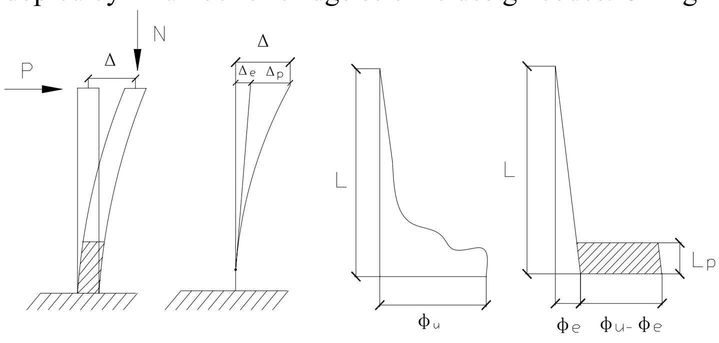


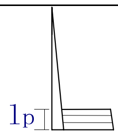
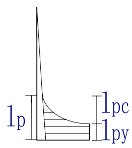
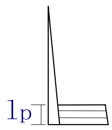
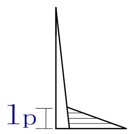
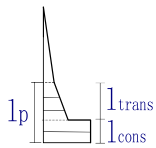

Fig. 1 Plastic hinge model of reinforced concrete column

mechanical effective plastic hinge length and parabola distribution of plastic curvature, and developed a computer program UB-COLA for hysteretic analysis of RC bridge piers under constant axial loading. Esmaeily and Xiao improved Priestley-Park model and Chang-Mander model by changing effective plastic hinge length and distribution form of plastic curvature in plastic hinge zone, and developed a computer program USC-RC for hysteretic analysis of RC bridge piers under variant axial loading. Berry and Eberhard analyzed trend of damage indices (such as concrete strain, plastic angle, drift ratio and displacement ductility) under variant damage levels with the change of bridge pier parameters such as axial compression ratio and shear span ratio.
With the development of performance-based/displacement-based bridge seismic design theory, higher demands for quantization of seismic damage indices are raised. Besides traditional displacement ductility and curvature ductility, residual displacement, maximum strain of longitudinal steel or concrete and low cycle fatigue damage indices of longitudinal steel are proposed to evaluate seismic damage of RC bridge piers. Up to now the degrees of accuracy to calculate foresaid damage indices with plastic hinge model have not been studied. Five commonly used plastic hinge models are employed to study the feasibility and main factor of the models for seismic damage evaluation of RC bridge piers by comparing the results of numerical analysis and test data. The purpose of this paper is to provide a more comprehensive basis of plastic hinge model in the application of performance-based bridge seismic design method and to promote the development of bridge seismic design codes.

## 2. SELECTED PLASTIC HINGE MODELS

Priestley-Park model, Chang-Mander model, Japanese code model, Esmaeily-Xiao model 1 and Esmaeily-Xiao model 2 are employed in the study, as shown in table 1.

Table1 Selected plastic hinge models

| Model | Effective plastic hinge model | Distribution form of plastic curvature | Diagrammatic sketch |
|---|---|---|---|
| Priestley-Park | $L_p = 0.08L + 0.022 f_y d$ | Rectangle |   |
| Chang-Mander | $L_p = L_{pc} + L_{py}$<br>$L_{pc} = L(1-\left\|\frac{M_y}{M_{\max}}\right\|)$<br>$L_{py} = 32\sqrt{d}$ | Parabola + Rectangle |   |
| Japanese code | $L_p = 0.2L - 0.1D$<br>$(0.1D \le L_p \le 0.5D)$ | Rectangle |   |
| Esmaeily-Xiao 1 | $L_p = L(1-\left\|\frac{M_y}{M_{\max}}\right\|)$ | Triangle |   |
| Esmaeily-Xiao 2 | $L_p = L_{trans} + L_{cons}$<br>$L_{trans} = 0.022 f_y d$<br>$L_{cons} = \begin{cases} D & (\lambda \le 12.5) \\ 0.08L & (\lambda > 12.5) \end{cases}$ | Ttrapezoid + Triangle |   |

Note: Lp=effective plastic hinge length，L=bridge pier height，fy=yield strength of reinforced steel，d=diameter of reinforced steel，D=diameter of bridge pier，My=initial yield moment of bridge pier，Mmax=Maximum moment

## 3. SELECTED BRIDGE PIER SPECIMENS

Twelve bridge pier specimens of low-cycle loading test are selected in the study accomplished by Lehman, Kunnath and authors respectively. The experimental data of Lehman and Kunnath were from PEER bridge pier database. The parameters of bridge pier specimens are shown in table 2.

Table 2 Parameters of bridge columns

| Specimen | Diameter (mm) | Height (mm) | Shear span ratio | Longitudinal ratio (%) | Stirrup ratio (%) | Axial compression ratio | Load pattern |
|---|---|---|---|---|---|---|---|
| LA407 | 609.6 | 2438.4 | 4 | 0.75 | 0.70 | 0.07 | Cyclic |
| LA415 | 609.6 | 2438.4 | 4 | 1.49 | 0.70 | 0.07 | Cyclic |
| LA430 | 609.6 | 2438.4 | 4 | 2.98 | 0.70 | 0.07 | Cyclic |
| LA815 | 609.6 | 4876.8 | 8 | 1.49 | 0.70 | 0.07 | Cyclic |
| LA1015 | 609.6 | 6096 | 10 | 1.49 | 0.70 | 0.07 | Cyclic |
| KA1 | 305 | 1372 | 4.5 | 2.0 | 0.87 | 0.1 | Monotonic |
| KA2 | 305 | 1372 | 4.5 | 2.0 | 0.87 | 0.1 | Cyclic |
| KA7 | 305 | 1372 | 4.5 | 2.0 | 0.87 | 0.1 | Random |
| KA8 | 305 | 1372 | 4.5 | 2.0 | 0.87 | 0.1 | Random |
| WA10 | 400 | 2450 | 6 | 1.5 | 0.5 | 0.19 | Cyclic |
| WA12 | 400 | 2450 | 6 | 2.2 | 0.36 | 0.19 | Cyclic |
| WA14 | 400 | 2450 | 6 | 2.9 | 0.36 | 0.19 | Cyclic |

## 4. MATERIAL CONSTITUTIVE RELATIONS

### *4.1 Reinforced steel*
Chang-Mander model for steel is a rule-based constitutive relation model, which has powerful ability to simulate strain stress relationship of reinforced steel and to describe the inelastic behaviors such as Baushinger effect, cyclic strain hardening, reversal memory and low cyclic fatigue etc. The model is composed of 10 rules, each describing strain stress path under certain conditions.

### *4.2 Concrete*
Chang-Mander model for concrete consists of 15 rules to simulate strain stress relation, which reflect the inelastic mechanisms of concrete such as softening effect, hardening effect and cracked surface effect. The rules of Chang-Mander model for reinforced steel and concrete are too complicated to introduce here, and the details can be seen in references.

## 5. ANALYSIS PROCEDURE OF PLASTIC HINGE MODEL

Displacement on the top of bridge pier can be figured out by means of relation between displacement and curvature along the pier. For example, the top displacement is obtained as equation 4 with rectangular distribution of plastic curvature.

$$\Delta = \Delta_e + \Delta_p = \frac{\phi_e L^2}{3} + (\phi - \phi_e) L_p (L - \frac{L_p}{2}) \tag{1}$$

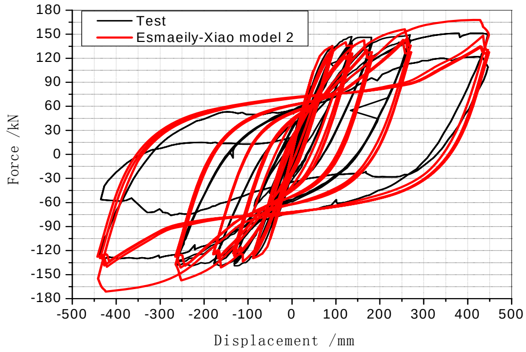
Where, Δ=displacement on the pier top; Δe=elastic displacement; Δp =plastic displacement; Φe=elastic curvature in the bottom cross section; Φ=total curvature in the bottom cross section; Lp=effective plastic hinge length; L=length from pier bottom to the center of upper mass.

Section curvature at bridge pier bottom is taken as load parameter and displacement on pier top is regarded as control parameter in the hysteretic analysis procedure of plastic hinge model. The program of seismic evaluation of RC bridge pier is as following: at first section analysis based fiber model should be performed to obtain the relation between moment and curvature of the bottom section, secondly displacement on pier top is calculated according to certain section curvature using plastic hinge model, and then relation between lateral force and top displacement is gained, at last the interested damage indices are computed and output.

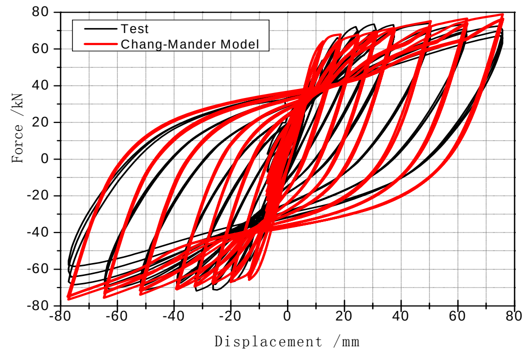
## 6. CALCULATION AND ANALYSIS

### *6.1 Hysteretic curves*

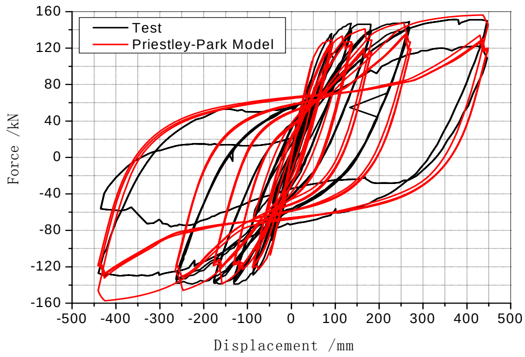
Computed hysteretic curves with selected models fit experiment results very well, even when the response of force and displacement of bridge piers approach strongly nonlinear stage, as can be shown in Fig. 2.



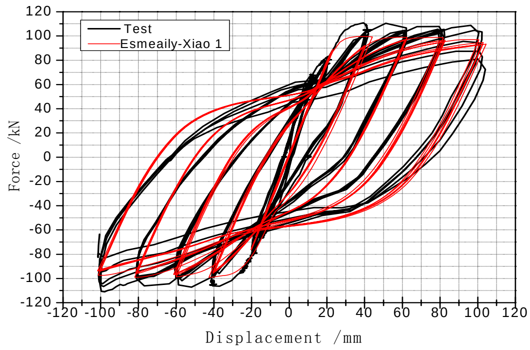

(a) KA2-Japanese code model




(b) KA2-Chang-Mander model




(c) LA815-Priestley-Park Model




(d) WA14-Esmaeily-Xiao model 1

Fig. 2 Comparison of the hysteretic curves between experimental and simulated results

### *6.2 Effect of bridge pier parameters*

*6.2.1 Maximum residual displacement*

Maximum residual displacement corresponds to maximum load control displacement when lateral force is equal to zero. Ratios of computed and experimental value of maximum residual displacement are shown in table 3, where the transverse average value and coefficient of variability mean statistic of one bridge pier corresponding to all models, the vertical ones mean statistic of one model corresponding to all bridge piers. As can be shown that computed results of five models are close, and that residual displacement is insensitive to variant models. Longitudinal steel ratio has little effect on residual displacement. The ratios decrease slowly with shear span ratio increasing. All the ratios are a little greater than 1.0 and are below 1.2. Residual displacements corresponding to other load control displacements are analyzed too, and the conclusion is similar to the maximum ones, as can be seen in the hysteretic curves.

*6.2.2 Ultimate curvature*

Ratios of computed and experimental values of ultimate section curvature at bottom of bridge piers are in table 4. Computed values of different models vary greatly from the minimum of Esmaeily-Xiao model 2 to the maximum of Japanese code model. Effective plastic hinge length is major impact on computed curvature, while the length of Esmaeily-Xiao model 2 has a lower limit of $D$ and Japanese code model has an upper limit of $0.5D$. The two models represent two extremities of effective plastic hinge length.

Table 3 Maximum residual deformation (computed value/test value)

| Specimen | Priestley | Chang | Japan | Esmaeily 1 | Esmaeily 2 | Average | Variability coefficient |
|---|---|---|---|---|---|---|---|
| LA407 | 1.24 | 1.20 | 1.25 | 1.17 | 1.07 | 1.19 | 0.05 |
| LA415 | 1.13 | 1.12 | 1.13 | 1.11 | 1.07 | 1.11 | 0.02 |
| LA430 | 1.17 | 1.17 | 1.17 | 1.17 | 1.13 | 1.16 | 0.01 |
| LA815 | 1.09 | 1.08 | 1.09 | 1.08 | 1.07 | 1.08 | 0.01 |
| LA1015 | 1.04 | 1.03 | 1.04 | 1.02 | 1.01 | 1.03 | 0.01 |
| Average | 1.13 | 1.12 | 1.14 | 1.11 | 1.07 | | |
| Variability coefficient | 0.07 | 0.06 | 0.07 | 0.06 | 0.04 | | |

Table 4 Ultimate curvature (computed value/test value)

| Specimen | Priestley | Chang | Japan | Esmaeily 1 | Esmaeily 2 | Average | Variability coefficient |
|---|---|---|---|---|---|---|---|
| LA407 | 1.48 | 1.11 | 1.76 | 1.02 | 0.86 | 1.25 | 0.26 |
| LA415 | 1.23 | 0.98 | 1.46 | 0.92 | 0.71 | 1.06 | 0.25 |
| LA430 | 1.84 | 1.53 | 2.19 | 1.47 | 1.08 | 1.62 | 0.23 |
| LA815 | 0.81 | 0.59 | 1.4 | 0.5 | 0.66 | 0.79 | 0.40 |
| LA1015 | 0.96 | 0.67 | 1.93 | 0.56 | 0.90 | 1.00 | 0.48 |
| WA10 | 1.41 | 1.3 | 1.87 | 1.3 | 0.91 | 1.36 | 0.23 |
| WA12 | 1.23 | 1.07 | 1.69 | 1.05 | 0.83 | 1.17 | 0.25 |
| WA14 | 1.62 | 1.41 | 2.38 | 1.4 | 1.16 | 1.59 | 0.26 |
| Average | 1.32 | 1.08 | 1.84 | 1.03 | 0.89 | | |
| Variability coefficient | 0.24 | 0.29 | 0.17 | 0.33 | 0.18 | | |

The effects of bridge pier parameters will be analyzed in the following. As to the piers with high shear span ratio such as LA815 and LA1015, computed curvatures are lower than experimental data except Japanese code model, which will result in a dangerous design of stirrups ratio in New Zealand code and will overestimate displacement ductility capacity in Caltrans and Chinese codes. So in seismic analysis of bridge piers of high shear span ratio using plastic hinge model, more attention should be paid into the effect of effective plastic hinge length. Longitudinal steel ratio has some effect on ultimate curvature too. Ratios of computed and experimental values of bridge piers with high longitudinal steel ratio are greater than those of bridge piers with low longitudinal ratio, for example, LA430 is greater than LA415 and LA407, WA14 is greater than WA10 and WA12. The conclusions about ultimate curvature are fit for curvature ductility factor based on the assumption that computed yield curvature is equal to experimental value.

At last, from comparison of average value and coefficient of variability of all the models, it can be concluded that Chang-Mander model and Esmaeily-Xiao model 1 have higher precision than the other three models. But considering composite factor such as suitability for high bridge pier, reliability and convenience for using, the authors recommend Priestley-Park model.

*6.2.3 Maximum tensile strain of outmost longitudinal steel*

Ratios of computed and experimental values of maximum tensile strain of outmost longitudinal steel at bottom section of bridge piers are shown in table 5. Most ratios are greater than 1.0, even 2.0 for Japanese code model. Ratios decrease with the increasing of shear span ratio, increase with the increasing of longitudinal steel ratio.

Table 5 The maximum tensile strain of longitudinal steel (computed value/test value)

| Specimen | Priestley | Chang | Japan | Esmaeily 1 | Esmaeily 2 | Average | Variability coefficient |
|---|---|---|---|---|---|---|---|
| LA407 | 1.58 | 1.20 | 1.96 | 1.08 | 0.95 | 1.35 | 0.27 |
| LA415 | 1.97 | 1.57 | 2.38 | 1.47 | 1.15 | 1.71 | 0.25 |
| LA430 | 1.98 | 1.64 | 2.48 | 1.58 | 1.24 | 1.78 | 0.24 |
| LA815 | 1.45 | 1.07 | 2.47 | 0.90 | 1.22 | 1.42 | 0.39 |
| LA1015 | 1.44 | 1.01 | 2.91 | 0.84 | 1.38 | 1.52 | 0.48 |
| Average | 1.68 | 1.30 | 2.44 | 1.17 | 1.19 | | |
| Variability coefficient | 0.24 | 0.26 | 0.30 | 0.30 | 0.14 | | |

From table 5 it is concluded that Chang-Mander model and Esmaeily-Xiao model 2 are more accurate than other models.

*6.2.4 Maximum compressional strain of core concrete*

Ratios of computed and experimental values of maximum compressional strain of core concrete at bottom section of bridge piers are shown in table 6. Except specimen LA430 with high longitudinal ratio, computed values are generally less than experimental data, with ratios about between 0.5 and 0.7. Ratios decrease with the increasing of shear span ratio and decrease with the increasing longitudinal steel ratio. It is concluded that the computed concrete strain by Japanese code model is closest to experimental data, and then Priestley, but the coefficient of variability approaches 0.5.

Table 6 The maximum compressive strain of core concrete (computed value/test value)

| Specimen | Priestley | Chang | Japan | Esmaeily1 | Esmaeily2 | Average | Variability coefficient |
|---|---|---|---|---|---|---|---|
| LA407 | 0.25 | 0.21 | 0.25 | 0.20 | 0.16 | 0.22 | 0.15 |
| LA415 | 0.71 | 0.55 | 0.79 | 0.55 | 0.29 | 0.58 | 0.30 |
| LA430 | 1.38 | 1.12 | 1.08 | 1.06 | 0.57 | 1.04 | 0.25 |
| LA815 | 0.62 | 0.45 | 0.96 | 0.41 | 0.53 | 0.59 | 0.33 |
| LA1015 | 0.46 | 0.36 | 0.81 | 0.29 | 0.46 | 0.48 | 0.38 |
| Average | 0.68 | 0.54 | 0.78 | 0.50 | 0.40 | | |
| Variability coefficient | 0.56 | 0.58 | 0.36 | 0.60 | 0.38 | | |

Table 7 Low cycle fatigue damage indices of outermost longitudinal steel-Kunnath fatigue-life equation

| Specimen | Priestley | Chang | Japan | Esmaeily1 | Esmaeily2 | Average | Variability coefficient |
|---|---|---|---|---|---|---|---|
| LA407 | 0.73 | 0.41 | 1.07 | 0.36 | 0.21 | 0.56 | 0.55 |
| LA415 | 1.95 | 1.23 | 2.87 | 1.08 | 0.56 | 1.54 | 0.52 |
| LA430 | 1.77 | 1.21 | 2.76 | 1.12 | 0.54 | 1.48 | 0.51 |
| LA815 | 0.73 | 0.41 | 2.6 | 0.29 | 0.46 | 0.90 | 0.96 |
| LA1015 | 0.66 | 0.34 | 3.24 | 0.23 | 0.56 | 1.01 | 1.12 |
| Average | 1.17 | 0.72 | 2.51 | 0.62 | 0.47 | | |
| Variability coefficient | 0.57 | 0.41 | 0.75 | 0.40 | 0.13 | | |

*6.2.5 Low cycle fatigue damage indices of longitudinal steel*

Computed low cycle fatigue damage indices of longitudinal steel with Kunnath equation is shown in table 7. Experiment by Lehman show that longitudinal steel fractured when specimens were failure except the buckled steel of LA430, while numerical simulation shows that computed damage indices of LA430 are almost the greatest among five bridge pier specimens. It is shown that average damage indices computed by all models vary from 0.5 to 1.5 except Japanese code model, but coefficients of variability approach 0.5-0.6. As to one single specimen, computed damage indices by different models have great difference too, especially coefficients of variability of LA815 and LA1015 are greater than 1.0.

***6.3 Effect of load path***

It is under monotonic loading that computing displacement capacity of bridge pier employing plastic hinge model according to common codes such as Caltrans and Japanese, which is different from quasi-static test with cyclic loading and from realistic earthquake. So the effect of load path on damage indices employing plastic hinge model is studied.

Table 8 Ultimate curvature (unit: $10^{-5}mm^{-1}$)

| Specimen | Priestley | Chang | Japan | Esmaeily 1 | Esmaeily 2 | Average | Variability coefficient |
|---|---|---|---|---|---|---|---|
| KA1_1 | 54.80 | 39.60 | 73.40 | 40.50 | 34.80 | 48.62 | 0.29 |
| KA1_2 | 27.50 | 21.40 | 36.20 | 22.30 | 17.70 | 25.02 | 0.26 |
| KA2 | 26.40 | 21.30 | 34.80 | 22.60 | 17.00 | 24.42 | 0.25 |
| KA7 | 27.20 | 21.10 | 35.80 | 22.60 | 17.75 | 24.89 | 0.25 |
| KA8 | 27.60 | 21.30 | 36.50 | 22.40 | 17.78 | 25.12 | 0.26 |
| Average | 27.18 | 21.28 | 35.83 | 22.48 | 17.56 | | |
| Variability coefficient | 0.02 | 0.01 | 0.02 | 0.01 | 0.02 | | |

Note： The load displacement of KA1_1 is 152mm, KA1_2 is 80mm.

Table 9 The maximum tensile strain of outermost longitudinal steel

| Specimen | Priestley | Chang | Japan | Esmaeily 1 | Esmaeily 2 | Average | Variability coefficient |
|---|---|---|---|---|---|---|---|
| KA1_1 | 0.1081 | 0.0784 | 0.1456 | 0.0801 | 0.0690 | 0.0962 | 0.29 |
| KA1_2 | 0.0549 | 0.043 | 0.0718 | 0.0448 | 0.0359 | 0.0501 | 0.25 |
| KA2 | 0.0593 | 0.0478 | 0.078 | 0.0508 | 0.0383 | 0.0548 | 0.24 |
| KA7 | 0.0551 | 0.0442 | 0.072 | 0.0446 | 0.0395 | 0.0511 | 0.23 |
| KA8 | 0.0573 | 0.0466 | 0.078 | 0.0465 | 0.0372 | 0.0531 | 0.26 |
| Average | 0.0567 | 0.0454 | 0.0750 | 0.0467 | 0.0377 | | |
| Variability coefficient | 0.03 | 0.04 | 0.04 | 0.05 | 0.04 | | |

Table 10 The maximum compressive strain of core concrete

| Specimen | Priestley | Chang | Japan | Esmaeily 1 | Esmaeily2 | Average | Variability coefficient |
|---|---|---|---|---|---|---|---|
| KA_1 | 0.0400 | 0.0287 | 0.0528 | 0.0293 | 0.0250 | 0.0352 | 0.29 |
| KA_2 | 0.0195 | 0.0148 | 0.0261 | 0.0155 | 0.0119 | 0.0176 | 0.28 |
| KA2 | 0.0107 | 0.0087 | 0.0137 | 0.0092 | 0.0072 | 0.0099 | 0.22 |
| KA7 | 0.0151 | 0.0119 | 0.0200 | 0.0122 | 0.0100 | 0.0138 | 0.25 |
| KA8 | 0.0121 | 0.0095 | 0.0180 | 0.0097 | 0.0078 | 0.0114 | 0.31 |
| Average | 0.0144 | 0.0112 | 0.0195 | 0.0117 | 0.0092 | | |
| Variability coefficient | 0.23 | 0.21 | 0.23 | 0.21 | 0.20 | | |

Computed ultimate section curvatures, maximum tensile strain of longitudinal steel and maximum compressional strain of core concrete are shown in table 8-10. Though computed damage indices by different models are different, as to one single model, load paths have little effect on curvature and steel strain, and coefficients of variability are 0.02 and 0.06 respectively. Load paths have a little effect on concrete strain. Computed concrete strain under monotonic loading is a little greater than those under random irregularly loading, while the latter is a little greater than those under cyclic loading. The perhaps reason is that constitutive relation model of steel is unable to describe buckling of steel.

## 7. Conclusion

A computer program for seismic damage analysis of RC bridge piers was developed adopting Chang-Mander

constitutive relation model with five common used plastic hinge models employed. Twelve bridge pier specimens of quasi-static test are simulated by the program, and effects of specimen parameters (such as shear span ratio and longitudinal steel ratio) and load path on damage indices (such as ultimate residual displacement, maximum strain of steel and concrete and low cycle fatigue damage index of steel) are analyzed. The main conclusions are as following:
(1) Computed hysteretic curves of RC bridge piers under cyclic loading by plastic hinge models fit experimental curves very well;
(2) Computed residual displacement by plastic hinge models is close to experimental data, and residual displacement is insensitive to variant plastic hinge models and to pier parameters;
(3) As to slender bridge piers with shear span ratio greater than 8, computed ultimate curvature by selected plastic hinge models is less than experimental data except Japanese code model.
(4) Plastic hinge models will overestimate maximum tensile strain of longitudinal steel and underestimate maximum compressional strain of core concrete;
(5) Statistic discreteness of computed low cycle fatigue damage indices by variant plastic hinge models are great, especially about slender bridge piers ($\lambda \geq 8$);
(6) As to one single model, load paths have little effect on curvature and steel strain and have a little effect on concrete strain.
Generally, Japanese code model is more conservative than other models, which is rational. As to this study, authors recommend Priestley-Park model prior, and then Chang-Mander model, but deep study of applicability for slender bridge piers should be performed. It is incomprehensive to access a plastic hinge model only by comparing simulated and experimental hysteretic curves. Limited to experimental data, analyzed damage indices mainly correspond to failure state of bridge piers. More study should be done about minor and middle damage states.

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