PERFORMANCE-BASED SEISMIC DESIGN OF RC BRIDGE PIER TAKING INTO ACCOUNT OF ENERGY DEMAND

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ABSTRACT:

For earthquake ground motion near active fault, there is enormous energy and displacement demand to structures. It is urgent to improve the existing seismic code and develop a new seismic design method which could consider simultaneously these two factors through performance-based design process. Combining the merits of response spectrum method and displacement-based method, inelastic displacement spectrum with expected performance target compatible with UBC97 seismic design code was deduced by the introduction of strength reduction factor with performance index as control parameter. On this basis, performance-based seismic design method for bridge pier was developed, which could put the expected performance index into design procedure effectively. Finally, the feasibility and effectiveness of this method was verified by practical RC bridge pier with various heights and expected performance targets.

KEYWORDS: performance-based seismic design, strength reduction factor, inelastic displacement spectrum, near-fault ground motion, performance index, bridge pier

1 INTRODUCTION

Since over the last 10 years, urban earthquakes close to the active fault zones (such as the 1995 Kobe earthquake in Japan, the 1999 Taiwan's Chi-Chi earthquake of China, etc.) have led to extremely heavy damage and hence drawn a high degree of concern from earthquake engineers all around the world. For the lessons from earthquake disaster and profound reflection of traditional method of seismic design, the Structural Engineers Association of California (SEAOC) Vision2000 Committee given the concept and theoretical framework of performance-based seismic engineering (referred to PBSE) and performance-based seismic design (referred to PBSD), which taken the research of Bertero (1995) as basis. In more than 10 years after this, great achievements have made: ATC-34(1995) issued by the U.S. National Earthquake Engineering Research Center (NCEER), Vision2000(1995) issued by the Structural Engineers Association of California (SEAOC), ATC-40(1996) issued by the California earthquake Safety Committee (CSSC), and the NEHRP guidelines FEMA-273 (1997) issued by the ATC and the U.S. construction Seismic Safety Association and the Federal Emergency Management Committee (FEMA), all of these files contain a great deal of ideas on performance-based seismic design. In China, the General rule for performance-based seismic design of buildings (2004) issued in 2004 also reflects such thought.

For earthquakes close to active fault, there co-exists high-energy and large deformation demand for civil structures and bridges, so it is necessary to consider both the two impacts on seismic performance. The research from Zhai Changhai and Xie Lili (2006) shows that in the range of short-period the strength reduction factors of near-fault records are significantly less than that of general earthquake records, which means it may be inappropriate for the existing strength reduction factors (which are from far-field earthquake records) applied to the near-fault region.

On the basis of the advised strength reduction factor (Jiang Hui, 2007), a performance-based seismic design method of bridge structure was developed in this paper, which could takes into account both energy and deformation demand of impulse-type earthquake motion.

2 PERFORMANCE TARGETS OF SEISMIC DESIGN FOR BRIDGE STRUCTURE

For performance-based seismic design of bridge structure, it is primary of all to make a clear delineation of performance targets. For the exact definition of performance indicators, Park. etc (Y.J.Park and A.H-S.Ang, 1985) proposed the relationship of damage index *DI* (by Park-Ang damage model of double-parameter) and destruction level of structures from the research of actual damage of 9 structures under different intensity of earthquake. And $DI = 0.4$ is taken as the critical value of repairable damage; when $DI > 1.0$, then the structure is under collapse. The relationship between damage index and state of structure is given in Table 1, which is used to define the expected damage performance of structure.

Table 1 Damage index (*DI*) proposed by Park & Ang

Damage index	State of structure
$DI<0.1$	Perfect or slightly damage
$0.1 \le DI<0.25$	Minor damage
$0.25 \le DI<0.40$	Moderate or repairable damage
$0.4 \le DI<1.0^*$	Severe or unrepairable damage
$DI \ge 1.0^*$	Collapse limit

* changed to 0.8 later

In order to consider both the demand of seismic fortification intensity and expected structure performance level, combined with the current seismic design code of China, Pan Long(2001) advised the damage goals of RC bridge structure under different levels of earthquake intensity (as shown in table 2).

Table 2 Damage goal for RC bridge under different earthquake intensity

type	Ⅰ（frequently）	Ⅱ（design）	Ⅲ（seldomly）
ⅠA		0～0.25	0.25～0.50
ⅠB	0～0.25	0.25～0.50	0.50～0.90
Ⅱ	0～0.25	0.25～0.50	0.50～0.90
Ⅲ	0～0.25		0.50～0.90

3 INELASTIC DISPLACEMENT DESIGN SPECTRUM BASED ON PERFORMANCE INDEX

In order to determine the natural vibration period of structure, it's necessary to give out the inelastic displacement design spectrum with performance index as control parameter. For the compatibility with current seismic design codes, an indirect method was used to calculate inelastic displacement spectrum, and the strength reduction factor of R_{DI} (Jiang Hui, 2007) was quoted here to obtain inelastic spectrum displacement:

$$R_{DI}(T, DI, \mu_u) = \frac{(e^{F_1 \cdot DI} - 1)(\mu_u - 1)^{F_2} T^{1.15}}{e^{F_3 \cdot DI} / (\mu_u - 1) + T^{1.15}} + 1 \tag{3.1}$$

In eqn.3.1, *DI* is the expected performance target of the structure, *T* is the natural vibration period of the structure, μ_u is the deformation ductile capacity under monotonic loading, and F_1, F_2, *and* F_3 are regression parameters. For single-degree of freedom system (SDOF), there is relationship between relative spectral displacement and absolute spectral acceleration as follows:

$$S_{de} = \frac{T^2}{4\pi^2} S_{ae} \tag{3.2}$$

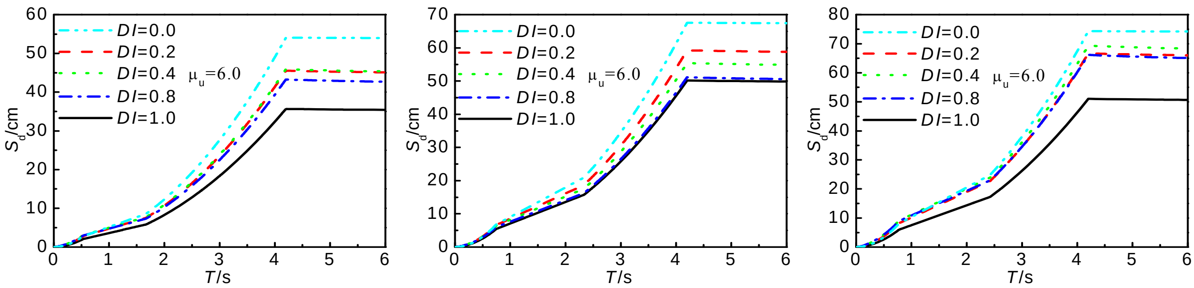
In which, S_{ae} and S_{de} are respectively elastic spectral acceleration and spectral displacement related to certain period and viscous damping ratio. For inelastic SDOF, there is an approximate relation between inelastic spectral displacement and elastic spectral acceleration:

$$S_a = \frac{S_{ae}}{R_{DI}} \tag{3.3}$$

Then combine eqn.3.2 and eqn.3.3, the inelastic displacement with exact performance target can be obtained as followed:

$$S_d = \frac{\mu_{eq}}{R_{DI}} S_{de} = \frac{\mu_{eq}}{R_{DI}} \frac{T^2}{4\pi^2} S_{ae} \tag{3.4}$$

In eqn.3.4, μ_{eq} (be defined in eqn.4.1) is equivalent ductile index according to certain damage state. In accordance with the UBC97 code of the United States, Figure 1 gives out the inelastic displacement spectrum for soil types of S_A, S_C and S_D and of different expected damage state. Since the effective period range for the advised R_{DI} is 0.2~4s, then the largest useful period of inelastic displacement spectrum is also 4s accordingly.



(a)UBC97 soil of S_A (b)UBC97 soil of S_C (c)UBC97 soil of S_D

Figure 1 Inelastic displacement design spectrum based on UBC97 with performance index (5% damping ratio)

4 PERFORMANCE-BASED SEISMIC DESIGN METHOD OF BRIDGE PIER

Seismic design method of structure has developed from a single-level fortification and one-stage design to dual-levels or three- levels fortification, two-stages or three-stages design, as well as to multi-levels fortification and multi-goals performance-based design process. In essence, performance-based seismic design is a further refinement of the consensus thought of multi-levels seismic fortification. By the combination of seismic fortification goal and design process, the structure would be more economic and reasonable, and the actual performance under different levels of ground motion would be controlled more accurately. At present, the most central issue for the performance-based seismic design method is how to implement the performance targets into the seismic design process.

In this paper, on the basis of the advised strength reduction factor R_{DI}, non-elastic displacement spectrum compatible with UBC97 seismic code was derived, and performance-based seismic design method of bridge structure was developed. Thus the performance targets could be integrated into the design process, rather than simply check after design.

4.1 Propose expected damage index (DI) of structure

First of all, it is necessary to determine the expected damage index *DI* of the bridge pier according to the importance of the structure, seismic fortification level and the needs of the owners. The lumped mass *m* above the bridge pier and the height *H* of the pier can be determined based on condition of structure layout and load. Then choose material parameters: axis compressive strength of concrete f'_c, steel yield strength f_y, reinforced concrete elastic modulus *E*, and determine the limit deformation capacity μ_u according to the type of material and structure. According to the improved Park-Ang damage assessment model, equivalent deformation ductility factor μ_{eq} (Jiang Hui, 2007) can be derived for different expected limit states:

$$\mu_{eq} = \frac{\sqrt{1+2\beta\chi\zeta'^2(DI\mu_\mu - DI + 1)} - 1}{\beta\chi\zeta'^2} \tag{4.1}$$

In eqn.4.1, χ is the ratio of hysteretic energy to input energy, ζ' is the equivalent velocity ratio of structure input energy to largest deformation. Eqn.4.1 can be used to characterize the equivalent ductility demand for certain damage index considering cumulative energy effect of near-fault ground motion.

4.2 Calculate target displacement Δ_d

Δ_d is related to the structure type of bridge and the design limit state (expected performance target). For the seismic design of bridge piers, the allowable displacement may be determined according to the geometric requirements (such as the supporting length), in order to ensure no pounding or opposite movement between two adjacent girders which would result falling of beams. Δ_d could be estimated in accordance with the formula presented by Calvi and Kingsley (1995) as follows:

$$\Delta_d = \delta H \tag{4.2}$$

In eqn.4.2, δ is drift ratio, and can be calculated by the followed formula:

$$\delta = \mu_{eq}\Phi_y H / 3 \tag{4.3}$$

And here

$$\Phi_y = \lambda\varepsilon_{sy} / D \tag{4.4}$$

In eqn.4.3, ε_{sy} is the yield strain of reinforcement and 2‰ is used commonly; *D* is the height of pier cross-section; λ is a dimensionless coefficient depending on the thickness of compression area and the yield curvature on nonlinear moment-curvature relationship, the value of 1.5 was advised by Calvi and Kingsley.

4.3 Determine the natural vibration period of bridge pier

According to the type of site soil, expected displacement limit Δ_d and design damage index *DI*, the fundamental natural vibration period T_e of bridge pier could be determined from the non-linear displacement response spectrum (Figure 1):

$$T_e = f(DI, \Delta_d, \xi) \tag{4.5}$$

4.4 Calculate design earthquake force

In response to the type of site soil and design fortification earthquake intensity, elastic design acceleration spectrum (such as design spectrum of UBC97 (1997) code in this paper) should be selected and reduced by inelastic reduction factor according to the expected performance index. Design shear force F_d and design moment M_d at the bottom of the pier can be calculated by the following equations:

$$F_d = m \cdot a_g \cdot \beta(T,\xi) / R_{DI}(T,\xi) \tag{4.6}$$

$$M_d = F_d \cdot H \tag{4.7}$$

4.5 Design cross-section and distribute steel

Inertia moment of RC pier cross-section is related to longitudinal reinforcement ratio and ratio of axial compressive force to axial compressive ultimate capacity of section, and cracking inertia moment I_{cr} is used in design (Kowalsky, Priestley, etc, 1995).

$$\frac{I_{cr}}{I_g} = 0.21 + 12\rho_l + (0.1 + 205(0.05 - \rho_l)^2)\frac{P}{A_c f_c^{'}} \tag{4.8}$$

In eqn.4.8, I_{cr} is the bending inertia moment of cracked section when tension reinforcement yields; I_g is bending inertia moment of gross section; ρ_l is longitudinal reinforcement ratio; $f_c^{'}$ is compressive strength of concrete; P is axial force; A_c is cross-section area of the pier. For piers with flexure as main deformation model, the I_{cr} could be drawn from the following formula:

$$I_{cr} = \frac{H^3}{3E} K_{eff} \qquad K_{eff} = \frac{4\pi^2 m}{T_e^2} \tag{4.9}$$

Once the design moment M_d is got, the geometry of cross-section could be estimated through a reasonable choice of ratio of axial compressive force to axial compressive ultimate capacity η_k:

$$\eta_k = \frac{P}{A_c f_c^{'}} \tag{4.10}$$

(a) Longitudinal reinforcement: According to the estimated geometry size of cross-section and ratio of axial compressive force to axial compressive ultimate capacity of section η_k, the longitudinal reinforcement ratio could be derived from the followed formula:

$$\rho_l = 0.05 - \frac{0.03}{\eta_k} + \frac{0.00244}{\eta_k}\sqrt{144 - 820\eta_k(0.81 + 0.1\eta_k - \frac{I_{cr}}{I_g})} \tag{4.11}$$

The economic reinforcement ratio by Kowalsky and Priestley(1995) is in the range of 0.7%≤ρ_l ≤4%；《Code for design technology of railway bridge and culvert》of China (1994) specifies that: The cross-section area of longitudinal reinforcement should not be less than 0.5 percent of the whole cross-section, and also nor more than 3 percent. It would be economic for the range of 0.8%～1.5%. If the longitudinal reinforcement ratio is not in such field, then the estimated geometry size of cross-section should be adjusted and recalculated.

(b) Hoop reinforcement: The hoop reinforcement ratio is relevant to the ductility requirement of cross-section. It could be calculated through sectional curvature ductility μ_φ, which is drawn by eqn.4.12 as a function of equivalent ductility factor μ_{eq}:

$$\mu_\varphi = 1 + \frac{(\mu_{eq} - 1)}{3(H_p / H)[1 - 0.5(H_p / H)]} \tag{4.12}$$

In eqn.4.12, H_p is the length of plastic hinge, which is influenced mainly by the development of plastic deformation and ultimate compression strain. It is determined primarily by experience as follows (Wang Dongsheng, 2002):

$$H_p = 0.08H + 0.022 f_y d_s \geq 0.044 f_y d_s \tag{4.13}$$

In the above formula, f_y and d_s are respectively the yield strength (N/mm^2) and diameter (m) of longitudinal reinforcement. The reinforcement distributing formula by Eurocode 8 code (1996) is as follows, which takes ductility factor μ_φ of section as a main parameter:

For rectangular cross-section:

$$\omega_\omega \geq 1.3(0.15 + 0.01\mu_\varphi)\frac{A_c}{A_0}(\eta_k + 0.08) \geq 0.12 \tag{4.14}$$

For circular cross-section:

$$\omega_\omega \geq 1.9(0.15 + 0.01\mu_\varphi)\frac{A_c}{A_0}(\eta_k + 0.08) \geq 0.18 \tag{4.15}$$

In eqn.4.14 and eqn.4.15, A_0 is the core area of concrete; ω_ω is mechanics hoop reinforcement ratio. The cubic hoop reinforcement ratio ρ_ω could be computed by the following formula:

$$\rho_\omega = \omega_\omega f_c' / f_y \tag{4.16}$$

The above formula takes into account the binding effect of hoop reinforcement, which could protect and strength the ductile deformation and energy dissipation capability of column plastic hinge.

4.6 Check and verification

Check the ductility design capacity, such as shear capacity of the cross-section, preventing buckling of longitudinal bar, as well as bending resistance capacity of non-plastic hinge regions.

5 DESIGN EXAMPLES

RC bridge piers would be designed according to the site soil S_C of UBC97 code, and the design fortification earthquake intensity is 9 degree (0.4g). The lumped mass *m* at the top of a pier is 200t; the domain of the piers' height is 6~15m; the axis compressive strength of concrete is 40Mpa; the yield strength of longitudinal bar is 400Mpa; the yield strength of hoop bar is 235Mpa; the elastic modulus *E* of reinforced concrete is 31.62Gpa; the damping ratio of the piers is 5%; and the thickness of covering layer takes is the 1/20 of the cross-section diameter. Set the expected performance targets of design piers as *DI*=0.2, 0.4, 0.6 and 0.8, which represents "minor damage", "medium damage", "serious damage" and "close to collapse" respectively.

For constant axial force, the design results of diameter and reinforcement bar corresponding to different pier height and selected performance targets are in table 3. Based on the method proposed, figures 2~5 gives out the seismic design outcome of bridge piers with circular section. Table 6 shows the hoop reinforcement ratio, and the mechanical hoop reinforcement ratios for all height and dimension are the same value of 0.180, which is the lower limit of code demand. It means at little ratio of axial compressive force to axial compressive ultimate capacity of section, the main deformation shape is flexure and detailing measure is enough. At fixed axial force, figure 2 gives the design moment for different pier diameters of variant heights. And it can be drawn that the design moment increases with the increment of pier diameter and height. Figure 3 reflects the change of longitudinal reinforcement ratio along with pier diameter. At the premise of a fixed axis, the higher the pier height, the larger pier diameter is needed for meeting the same performance state. And the longitudinal reinforcement ratio decreases rapidly with the reduction of pier diameter. Figure 4 shows the design moment for different performance levels. With the augment of expected performance level, the pier is allowed higher earthquake damage. And the tendency is the identical for different pier heights. Figure 5 discusses the variety of pier diameter with the change of design performance index. With the increment of *DI*, the pier diameter declines for lower earthquake demand, which reflects the rationality of performance-based seismic design.

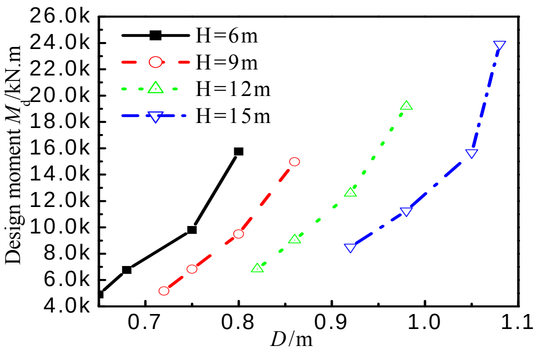


Figure 2 Design moment vs. pier diameter

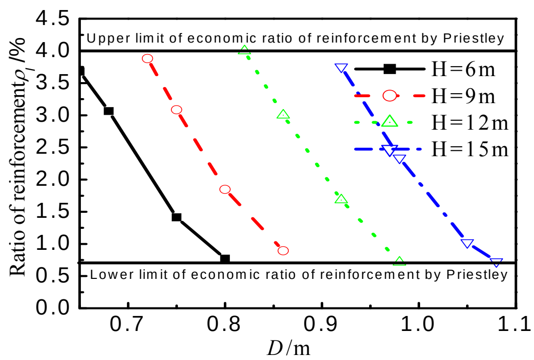


Figure 3 Longitudinal ratio of reinforcement vs. pier diameter

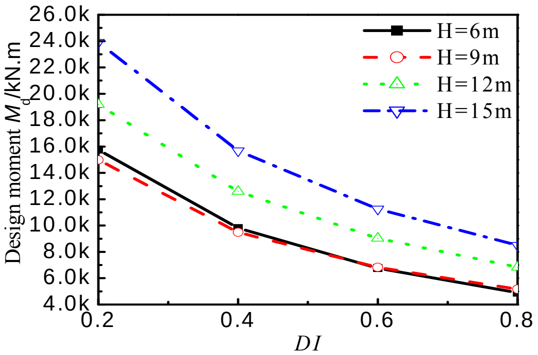


Figure 4 Design moment vs. *DI*

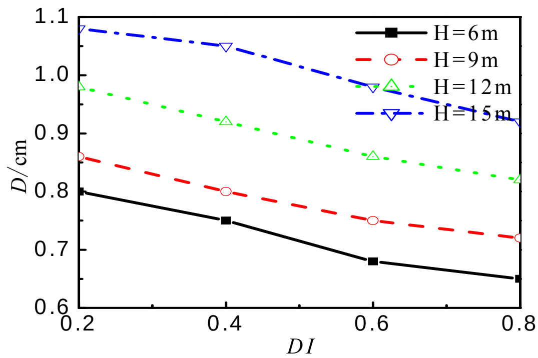


Figure 5 Pier diameter vs. *DI*

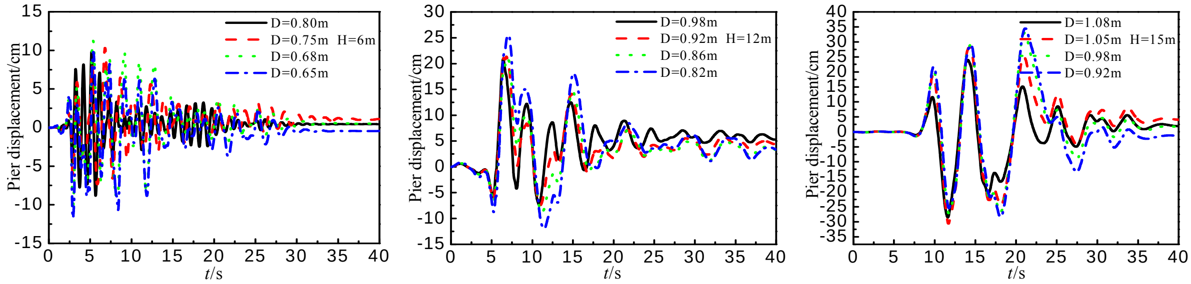
Table 3 Result of steel distributed for bridge piers with different height and *DI*

H/m	*DI*	μ_{eq}	T_e/s	$\beta(T)$	R_{DI}	M_d /kN.m	D/m	η_k	L_p/m	ρ_l/%	ω_ω
6	0.20	1.409	1.45	0.61793	1.808	15754.3	0.80	0.097	0.700	0.771	0.180
	0.40	2.567	1.54	0.58182	2.737	9802.2	0.75	0.111	0.675	1.412	0.180
	0.60	3.574	1.61	0.55652	3.800	6758.0	0.68	0.135	0.640	3.066	0.180
	0.80	4.476	1.68	0.53333	5.016	4898.9	0.65	0.148	0.625	3.680	0.180
9	0.20	1.409	2.25	0.39822	1.838	14975.0	0.86	0.084	0.880	0.893	0.180
	0.40	2.567	2.36	0.384	2.798	9491.7	0.80	0.097	0.850	1.848	0.180
	0.60	3.574	2.41	0.384	3.892	6822.9	0.75	0.111	0.825	3.084	0.180
	0.80	4.476	2.46	0.384	5.141	5164.6	0.72	0.120	0.810	3.880	0.180

12	0.20	1.409	2.66	0.384	1.846	19178.6	0.98	0.065	1.090	0.716	0.180
	0.40	2.567	2.74	0.384	2.813	12585.1	0.92	0.074	1.060	1.682	0.180
	0.60	3.574	2.80	0.384	3.918	9037.2	0.86	0.084	1.030	3.001	0.180
	0.80	4.476	2.86	0.384	5.179	6835.9	0.82	0.093	1.010	3.999	0.180
15	0.20	1.409	2.98	0.384	1.851	23910.9	1.08	0.054	1.290	0.727	0.180
	0.40	2.567	3.07	0.384	2.824	15673.3	1.05	0.057	1.275	1.022	0.180
	0.60	3.574	3.13	0.384	3.934	11248.5	0.98	0.065	1.240	2.338	0.180
	0.80	4.476	3.19	0.384	5.203	8505.6	0.92	0.074	1.210	3.753	0.180

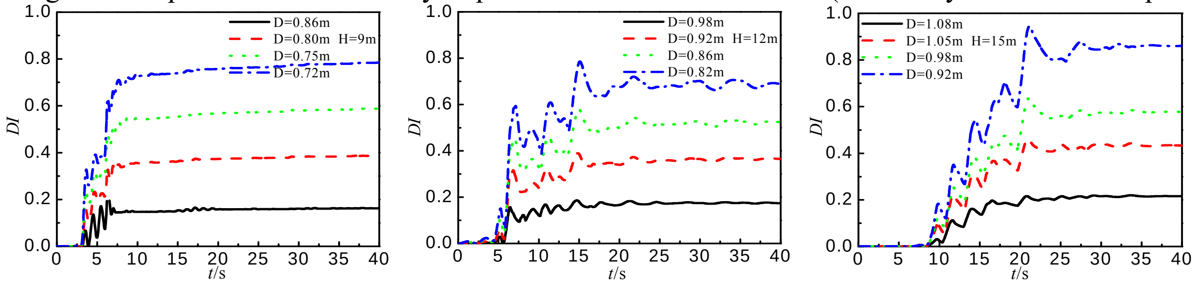
6 VERIFICATION OF THE EXAMPLE

Two groups of earthquake records (E1, E2) (Jiang Hui, 2007) was used to check the effectiveness of the proposed method. Inelastic dynamic analysis was introduced to verify the seismic damage of bridge piers with selected performance state. Figure 6 gives out the displacement time history of piers with height of 6m, 12m and 15m respectively. For certain height, the peak displacement is different obviously for corresponding performance level, which reflects its control role for seismic design. When DI = 0.60, it can be seen from the figure that the maximum elastic-plastic displacement is 0.114 m, 0.226m and 0.337 m for each pier with diameter of 0.68m, 0.82m and 0.92m. With the exception of an 11.4 percent ultra limit of displacement for the pier with height of 15m, other piers do not exceed the displacement limits, meeting the engineering requirements. Figure 7 describes the damage time history of piers with height of 9m, 12m and 15m for different performance levels. Table 4 lists earthquake damage contrast from dynamic time history analysis for piers under the actual seismic records (E1) and artificial pulse (E2). For the group of actual records(E1), with the exception of the pier of 15m (design DI=0.80), whose actual average earthquake damage is slightly higher (9.4 percent) than the design target , the other piers all meet the expected performance demand and tend to approach the design value as the of height increases. For another group of artificial pulse (E2), since the impulse period T_P is close to the natural vibration period T_e of structure, the actual damage for piers of 12m and 15m is a little larger than the expected value, but no more than 15 percent of the expected value.
Overall, the design method advised in this paper can generally guarantee the earthquake damage of each pier not exceed the expected value, whether for actual records or artificial impulse. And the calculated average damages are close for the two groups.



(a) (b) (c)

Figure 6 Displacement time history of piers with different diameters (excited by E1 artificial impulse)



(a) (b) (c)

Figure 7 Damage time history of bridge piers

Table 4 Dynamic time history check result of piers under real records and artificial impulse

H/m	Design *DI*	*D*/m	Real records（E1）		Artificial impulse（E2）	
			Mean *DI*	Deviation ratio %	Mean *DI*	Deviation ratio %
6	0.20	0.80	0.1796	10.2	0.1804	9.8
	0.40	0.75	0.3536	11.6	0.358	10.5
	0.60	0.68	0.5034	16.1	0.5094	15.1
	0.80	0.65	0.6784	15.2	0.6768	15.4
9	0.20	0.86	0.1796	10.2	0.1901	5.0
	0.40	0.80	0.3664	8.4	0.3592	10.2
	0.60	0.75	0.537	10.5	0.5424	9.6
	0.80	0.72	0.7032	12.1	0.7	12.5
12	0.20	0.98	0.1856	7.2	0.1828	8.6
	0.40	0.92	0.362	9.5	0.362	9.5
	0.60	0.86	0.5328	11.2	0.5262	12.3
	0.80	0.82	0.7008	12.4	0.8912	**-11.4**
15	0.20	1.08	0.1836	8.2	0.1754	12.3
	0.40	1.05	0.3692	7.7	0.4564	**-10.1**
	0.60	0.98	0.531	11.5	0.7092	**-12.2**
	0.80	0.92	0.8752	**-9.4**	0.9632	**-14.1**

7 CONCLUSIONS

For earthquake ground motion close to active faults, there is huge energy and displacement demand to structures. It's essential to develop a new seismic design method which could consider simultaneously these two factors by the way of performance-based design process. Based on the strength reduction factor with performance index as control parameter, the inelastic displacement spectrum compatible with UBC97 Seismic Design code was educed. Then performance-based seismic design method for bridge pier was developed, which could put the expected performance target into design procedure effectively. Finally, examples of bridge pier were designed to verify its feasibility and effectiveness. Time history analysis results show that the actual damage of RC bridge piers designed following this procedure can be controlled effectively within expected scope.

ACKNOWLEDGMENTS

This research was supported by National Natural Science Foundation of China under Grant No. 50278002 and 50578007, and also got the support of Science Fund of Beijing Jiaotong University under Grant No. 2008RC026.

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