ASSESSMENT OF SEISMIC DESIGN FOR BRIDGE IN VIETNAM - A LOW MODERATE SEISMIC ZONE

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This paper provides a brief review seismic design guideline for bridge in Vietnam and calculates response of bridge in a low to moderate magnitude such as Vietnam (i.e. acceleration coefficient, A= 0.00 - 0.29g). Under strong ground excitations, response of a bridge shows severe nonlinearity induced by inelastic deformation at a plastic hinge of a pier. Seismic design guidelines in the current Vietnam indicate that ground acceleration of 0.09 g or greater is likely to produce nonlinear structural response. An empirical beam seat formula indicates the upper limit of displacements due to response, since the requirement of its is predicted on the assumption of plastic response. This study is also to determine a hysteretic curvature at the plastic hinge of a pier and further to examine whether the empirical displacement formula is suitable for bridges in those seismic zones.

Key Words: *Seismic analysis, seismic zone, displacement, response analysis, bridge*

1. INTRODUCTION

Recently, the strong earthquake such as Northridge Earthquake of 1994, the Hyogoken-Nanbu Earthquake of 1995, the Taiwan Chi-Chi Earthquake of 1999, the Iran Earthquake of 2001, the Chuetsu Earthquake of 2004, and the Wengchuan Earthquake of 2008, have caused serious damage to many lifeline facilities, including bridges.

Vietnam has not experienced any big earthquake damages and the history of large scale earthquakes up to now. The seismic designs for the bridges weren't adopted until former 90's. Many seismometers are installed and some middle scale earthquakes have been recorded in Vietnam. According to the analyses of the earthquake records obtained by the seismometers, Vietnam is located at a moderated seismic activity area. However, seismic design for buildings and bridges become to be regarded as to be important in Vietnam. Specifications for bridge design in Vietnam [1)] (referred to as 22TCN-272-05) was established on base of AASHTO LRFD 1998 [3)] in 2001, officially applied in 2005. As for an essential multi-spans bridge located moderate seismic zone, the seismic design can be conducted by single-mode elastic method or uniform load elastic method according to Vietnam Specification [1)].

This paper describes a brief review on seismic design for bridge in Vietnam specification. Then, parametric studies involving a continuous multi-span bridge in Vietnam is followed. Like this, the other objective of this study is to determine response seismic of the pier bridge at the plastic zone in a low to moderate seismic zone (i.e. acceleration coefficient, A = 0.09 – 0.29g with g being the gravity acceleration), and to make sure that the empirical beam seat formula is appropriate for typical bridge in Vietnam.

2. BRIEF REVIEW ON SEISMIC DESIGN FOR BRIDGE IN VIETNAM SPECIFICATION

(1) General

The current Vietnam Specifications of bridge design published in 2005 was established according to AASHTO LRFD 1998. This specification is also concerned with seismic design. In this code, some objects are modified according to Vietnam conditions. Namely the map of maximum seismic intensity zone [1], the map of a acceleration coefficient as shown in **Fig. 1** [2], the seismic zones which is classified into three seismic zones as presented in **Table 1** [1], the acceleration coefficients which are adopted 0.00 to 0.29 g, etc., are modified. However, the concept of the structure analysis and seismic analysis are originally taken from AASHTO LRFD 1998 [3].

(2) Response and design spectra

When a structure responds to an applied large scale earthquake load and live load, the corresponding displacement may be large enough to induce nonlinear deformation. The response spectra for elastic behavior are fairly different than those for nonlinear behavior. The equations and provisions specified in the design codes are based entirely on elastic behavior analysis [1, 3, 4].

The earthquake load shall be taken to be horizontal force effects determined by the product of the mass, the response modification factor R shall be taken from 0.8 to 5.0 to depend on importance categories and bridge components, and the elastic seismic response coefficient C_{sm} for the mth vibration mode. The elastic seismic response coefficient may be normalized using the input ground acceleration (A) and the result plotted against the period of vibration. This coefficient is given as following equations [1, 3, 4]:

$$C_{sm} = \frac{1,2AS}{T_m^{\ 2/3}} \le 2,5A \qquad (T_m \le 4.0 \text{ s}) \qquad (1a)$$

$$C_{sm} = \frac{3AS}{T_m^{4/3}} \qquad (T_m > 4.0 \text{ s}) \qquad (1b)$$

and soil profiles type III, IV:

$$C_{sm} = A\,(0.8 + 4.0T_m) \qquad (T < 0.3 \text{ s}) \qquad (1c)$$

where T_m is the period of the mth vibration mode (s); A is the acceleration coefficient and it is determined in accordance with the map of seismic zones and maximum seismic intensity zone of Vietnam (reference to **Fig. 1**) and, it was given by the Vietnam Institute of Geophysics and provided as contour for return period of 500 years. Maximum probable earthquake with a return period of around 2.500 years has to be considered with the critical bridges. S as shown in **Table 2** is the site coefficient.

The structure's period will render the bridge engineer idea on the performance of the structure through the design response spectra. If the period is large, it will likely fall with the dominant displacement portion of the spectra and displacement of the structure will be large. Current industry practice, due largely the development of computer hardware and software, is to perform a multi-modal response spectrum analysis on the bridges. The displacement demands are then determined directly from the analysis results. It should be recognized that these values will be inherently conservative due to the nature of the design response spectra [7].

The four descriptive soil types are defined as follows [1, 3, 4]:

+ Type I (S = 1.0): Rock of any characteristic or any stable deposit of sands, gravels, or stiff clays less than 60 m deep and overlying rock.

Table 1 Seismic zone in Vietnam

Acceleration Coefficient	Seismic zone	MSK - 64 class
$A \le 0.09$	1	Class ≤ 6.5
$0.09 < A \le 0.19$	2	6.5 < Class ≤ 7.5
$0.19 < A < 0.29$	3	7.5 < Class ≤ 8

Table 2 Site coefficients (22TCN 272-05)

Site coefficient	Soil profile type			
	I	II	III	IV
S	1.0	1.2	1.5	2.0

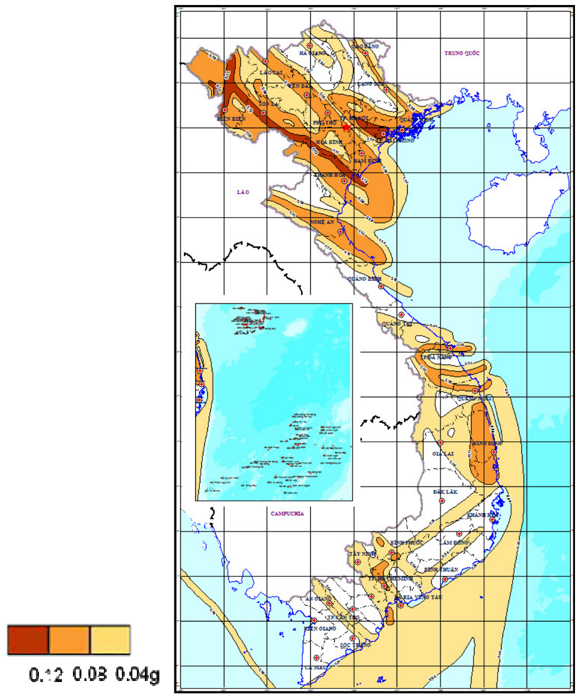

Fig. 1 Ground acceleration zone map of Vietnam with return period about 500 years [2]

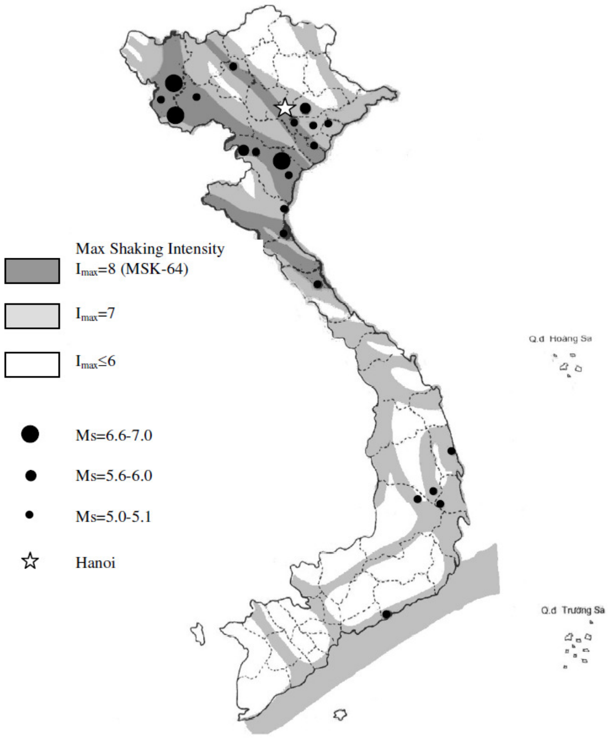

Fig. 2 Ground acceleration zone map of Vietnam with return period about 1000 years [6)]

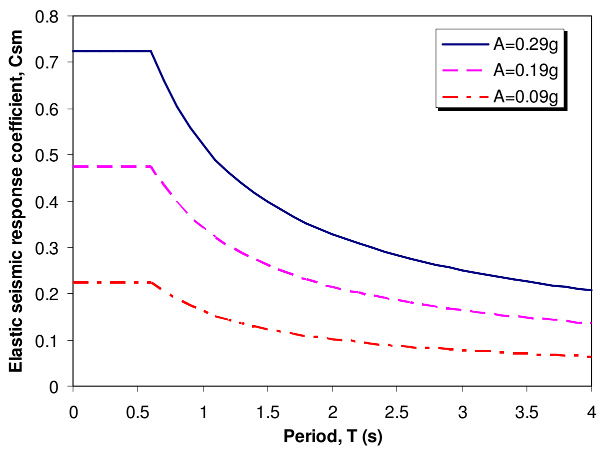

Fig. 3 Design response spectrum

+ Type II (S = 1.2): Deep cohesion-less soil including any stable deposit of sands, gravels, or stiff clays greater than 60 m deep and overlying rock.

+ Type III (S = 1.5): Soft to medium stiff clay, sand, or other cohesion - less soil generally greater than 9 m deep.

+ Type IV (S = 2.0): Soft clays or silts greater than 12 m in depth.

In the seismic design in Vietnam, except for some especial bridges, the ordinary bridges are design by static method. The design sectional forces and displacements are calculated by acting the earthquake load as static force and using liner static analysis on a beam model. But as for the important especial bridges such as cable stayed bridge, long span bridge and the some bridges have been constructed by Official Development Assistance financial support (ODA) for infrastructure development projects in Vietnam, etc, the seismic design is conducted by Japan Specifications for Highway Bridge (referred to as JRA-2002) to secure more safety of the bridges.

(3) Plastic hinge

When seismic forces are estimated from an elastic analysis, 22TCN 272-05 and AASHTO LRFD1998 allow these forces to be reduced by appropriate response modification factors. These reduced forces can be used for design, but only if the substructure units are made ductile enough to undergo plastic hinge with out suffering catastrophic failure. The philosophy is that seismic induced forces can only become large enough to produce plastic hinges. Once plastic hinges form, force can no longer be absorbed, but deflections will be large [7)]. The response modification factors are applied to forces and not displacements. Since the codes require the seismic analysis and ductile details and allow the use of response modification factors for typical bridge structures with seismicity higher than seismic zone 2 [1), 3), 4)] (i.e. acceleration coefficient A > 0.09 g), it is apparent that plastic behavior would be a real possibility in these regions.

In the both code have adopted a policy aiming at preventing catastrophic bearing seat loss failure if the elastic limit of the piers is exceeded. This policy requires that bearing seat lengths be constructed long enough to accommodate the maximum displacements obtained from an elastic analysis of the structure or those obtained from the beam seat length (N) shall be taken empirical displacement formula as follows:

$$\text{Minimum seat width} = f \times N \qquad (2a)$$

where f is factor based on seismic performance category and is expressed as a percentage; and N is minimum support length measured normal to the centerline of bearing (mm) expressed as

$$N = (200+0.0017L+0.0067H)(1+0.000125S^2) \qquad (2b)$$

where L is length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck; for hinges within a span, L shall be the sum of the distances to either side of the hinge; for single-span bridges, L equals the length of the bridge deck (mm); H is average height of columns supporting the bridge deck to the next expansion joint (mm) for columns and/or piers, column, or pier height (mm) for hinges within a span, average height of the adjacent two columns or piers (mm) for single-

Table 3 Acceleration coefficients of some bridges have been constructed in Vietnam

No	Name of Bridge	Typical Bridge	Span Layout	MSK-64 Class	Acceleration coefficient
1	Tan De	Cantilever Bridge	75+3@120+70	8	0.10
2	Phu Dong	Cantilever Bridge	65+7@100+65	7(8)	0.17
3	Bai Chay	Cantilever Bridge	40+81+129+435+129+86	6	0.17
4	Kien	Cable stayed Bridge	85+200+85	7	0.06
5	Can Tho	Cable stayed Bridge	2@40+150+550+150+2@40	6	0.10
6	Thanh Tri	Cantilever Bridge	80+4@130+80	8	0.17
7	Da Bac	Cantilever Bridge	65+100+65	7	0.07
8	Quy Cao	Cantilever Bridge	52+85+52	7	0.08
9	Non Nuoc	Cantilever Bridge	42+52+85+52+42	7(8)	0.10
10	Tram Bac	Cantilever Bridge	52+85+52	7	0.07
11	My Thuan	Cable stayed Bridge	150+350+150	6	0.10
12	Ben Luc	Cantilever Bridge	50+90+120+90+50	7	0.10
13	Nhat Tan	Cable stayed Bridge	150+4@300+150	7(8)	0.12
14	Phu Long	Cantilever Bridge	75+120+75	7	0.08
15	Dong Tru	CFST arch bridge	80+120+80	8	0.17
16	Rao II	Cable stayed Bridge	-	-	0.14
17	Phap Van-Cau Gie Interchange	PC beam Bridge	29+29.5+3@30+29.5+29	-	0.17
18	Hoa Binh	PC beam Bridge	-	-	0.19

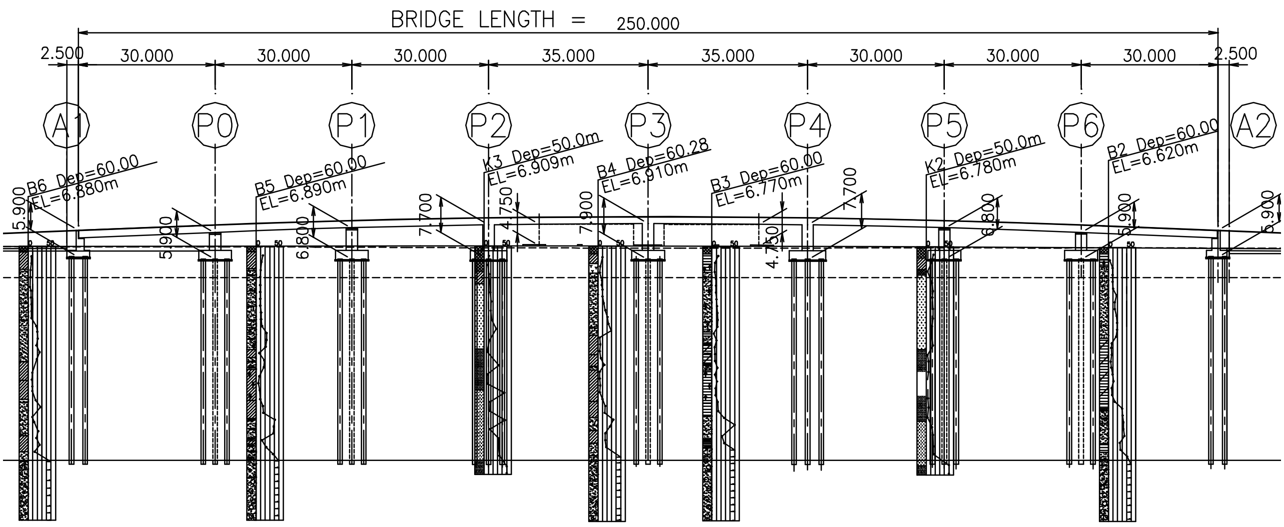

Fig. 4 The profile of the highway bridge (unit is mm)

-span bridges (mm); S is skew of support measured from line normal to span (deg).

The equation (2) has been taken originally from AASHTO LRFD 1998 and it can often give displacement several times larger than those obtain from an elastic analysis.

3. PARAMETRIC STUDIES

(1) Bridge model

The profile of multi-span continuous bridge was shown in **Fig. 4**. Representative of typical bridges in Vietnam is evaluated under this study. This bridge in this study is designed by static analysis according to the 22TCN-272-05 [1)]. The superstructure is a hollow concrete slab beam structure with 8 continuous spans. The total length of the bridge is 250 m.

The substructure system consists of 3 rigid frame piers (P2, P3, and P4) and 4 bent piers (P0, P1, P5 and P6). The compressive strength of the concrete of all piers is 30 MPa; the diameter of the spiral reinforcement is 16 mm; the diameter of the longitudinal reinforcement is 25 mm and 29 mm, spacing of the spiral is 300 mm and 125 mm for P0, P1, P5, P6 and P2, P3, P4, respectively. The rubber bearing supports (P0, P1, P5 and P6) are installed. The basic components of the rubber bearing are elastomer and steel plates. All of the pier columns and the abutments are fabricated by reinforced concrete and the bored cast-in-place piles are driven under the footing as shown in **Fig. 5**. The pier columns are all circular with spiral or circular lateral reinforcement as shown in **Fig. 6**. The ground layer consists of medium sand, fine sand and gravelly sand. The thickness of the surface layer is about 40 m.

(2) Analysis procedure

(a) Model

The bridge in this study is designed by the Vietnam design code by static analysis. The stopper and rubber bearing are installed to enhance the seismic performance. In this study, the response analysis of the bridge is estimated according to JRA-2002 [5)]. An analytical model of the bridge as shown in **Fig. 7** is made to estimate seismic performance of the bridge, especially bearing capacity of the members and unseating of the girders. The girders are replaced to linear beam elements. The pile foundation is replaced to the horizontal spring K_x, vertical spring K_y and rotating spring K_θ. The spring values are calculated by Forum 8 software. The dominate period of the surface ground is $T_G = 0.88$ s > 0.60 s i.e. ground type in **Table 4** is type III ground. The modified factor C_z is selected as 0.7 (region C). The concrete block as a stopper is installed at the top of the pier. The stopper is replaced to a spring element considering the spacing. The rubber bearing is replaced to a bi-linear spring element in horizontal direction.

The pier has circler cross section as shown in **Fig. 6**. The compressive strength of the concrete is 30MPa, the diameter of the spiral reinforcement is 16mm and spacing of the spiral is 300mm. The column of the pier is replaced to nonlinear beam elements. The nonlinear behavior of the columns is presented by the Takeda model with the potential plastic hinge zone located at bottom of the column. The Takeda hysteresis property is adopted for bending deformation of the pier. Relationship of M - ϕ and M - θ for column piers are used for this analysis. The stress vs. strain relation of reinforcing bars is idealized by bi-linear model. In the pier column, a plastic hinge modeled by nonlinear rotating spring is arranged at the bottom column zone.

The length of the plastic hinge is calculated according to JRA-2002 [5)], equation as follows:

$$L_p = 0.2h - 0.1D \qquad (3a)$$

in which:

$$0.1D \leqq L_p \leqq 0.5D \qquad (3b)$$

where L_p is plastic hinge length; h is height of the column pier; D is section depth.

The Rayleigh damping coefficients are calculated from the vibration frequencies of the structure. Natural dominate frequencies of the structure are 2.337 H_z and 7.305 H_z. The values of the Rayleigh damping coefficients are α = 0.45829 and β = 0.00225, respectively. The commercial finite element analysis program (TDAP III software) is used for the analysis. The numerical integration is performed using the Newmark-β method and integration time interval is 0.01s.

Table 4 The characteristic value of the surface ground

Layer	h_i (m)	SPT (N)	γ_t (kN/m^3)	V_{si} (m/s)	$T_i = H_i/V_{si}$ (s)
Silt clay	5.9	3	17.4	138.67	0.04
Sand	10.4	10	16.0	218.28	0.05
Lean clay	3.4	3	18.3	135.72	0.03
Fine sand	5.3	21	16.8	273.69	0.02
Lean clay	5.1	3	16.5	144.22	0.04
Lean clay	2.9	12	19.7	228.94	0.01
Sand	4.0	20	16.8	271.44	0.01
Fine sand	2.9	24	16.8	288.45	0.01
Pebble	3.0	50	21.0	368.40	0.01

The characteristic value of the ground:
$T_G = 4 \cdot \Sigma T_i = 0.88$ (s) > 0.60 (s)
→ The ground type is type III according to JRA-2002 [5)]

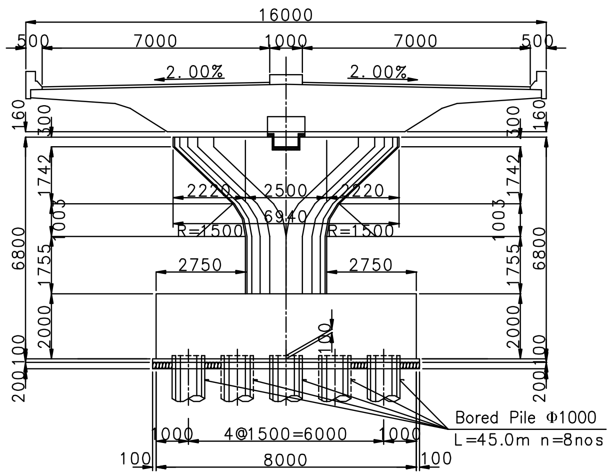

Fig.5 The profile of the pier and the piles (unit is mm)

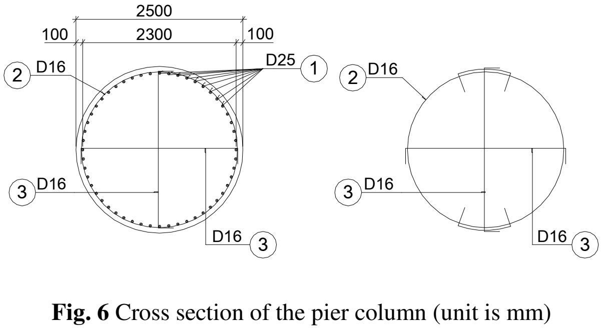

Fig. 6 Cross section of the pier column (unit is mm)

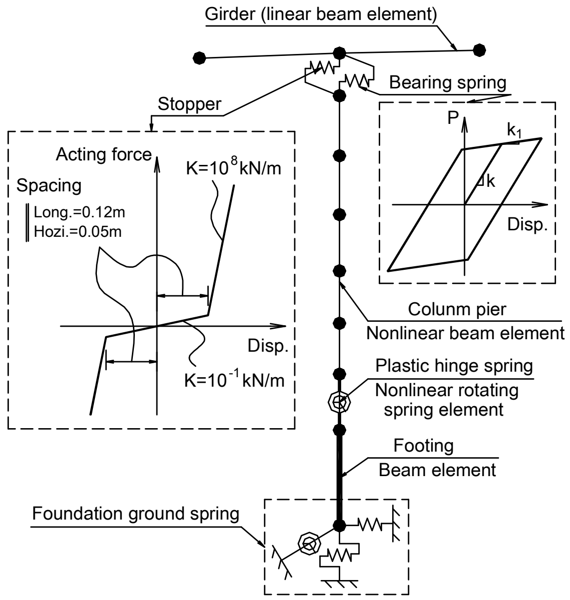

Fig. 7 Modelling of the bridge pier

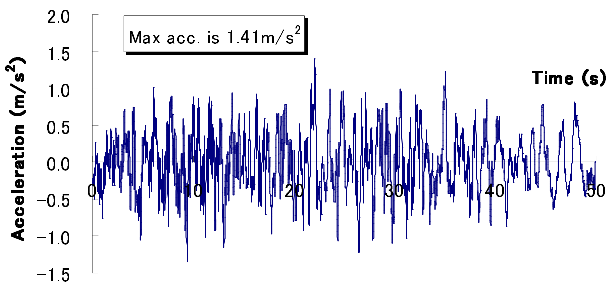

Fig. 8 The ground acceleration records in the Tsugaru Ohhashi, 1983 (M_g = 7.7, the maximum acc. is 1.41 m/s^2)

Table 5 The displacement of the top pier (unit is m)

Pier	P0	P1	P2	P3	P4	P5	P6
	0.002	0.007	0.013	0.013	0.013	0.007	0.002

Table 6 Moment and shear force at the plastic hinge

Pier	Shear (kN)		Moment (kN·m)
	Response	Resistance	
P0	502.4	3854	1738
P1	1588	3854	6914
P2	4605	5229	6977
P3	4621	5229	7099
P4	3986	5229	8210
P5	1597	3854	6953
P6	505.9	3854	1751

(b) Ground motion

Base on research projects of the Vietnam Institute of Geophysics named "Research and Forecasting Earthquakes and Foundation Fluctuations in Vietnam" reported recently (Nguyen Dinh Xuyen et al., 2005) and other researchers [6), 8)] shown that Vietnam was classified to the low moderate seismic zone, the maximum magnitude do not exceed 6.0 on the Richter scale, but the earthquakes of magnitude (M_S) greater than 3.1 on the Richter scale occurred in Vietnam. The focal depth of most earthquakes is 10-20 km. Most of earthquakes did not cause any serious damage for structures, especially for bridge structures. However, since bridge damage due to the design earthquake could influence on the current life, transportation and economy, the current design code require seismic analysis for bridge into 3 seismic zones with acceleration coefficients from 0.00 to 0.29 g shown in **Table 1**. The ground acceleration records in the Tsugaru Ohhashi (1983), Japan is adopted as input data at the ground level in this study (shown in **Fig. 8**). This record is corresponded to Level 1 ground motion in Japan Specification, JRA-2002 [5)] and maybe corresponds with a low moderate seismic zone in Vietnam [6)]. The peak ground acceleration of earthquake wave is 1.41 m/s^2. These ground acceleration records are adopted because the soil condition of the construction site is classified into Group III in the soil condition.

(3) Results and evaluation

(a) Response of the piers bridge

The results show the maximum rotation angles of the pier, shear force and moment obtained from analysis are smaller than there resistance (shown in **Table 5** and **Table 6**). **Fig. 9** to **Fig. 15** shows the hysteretic response at the plastic hinge of the pier including the rubber bearing is installed at the top of the pier P0, P1, P5 and P6, and rigid jointed between the pier and the girder at the pier P2, P3 and P4. The displacement at the top of the pier is also small. The analysis shows that all piers are still within elastic state, no serious damage is evaluated for Level 1 earthquake motion. The bridge is secure from the current earthquake occurring in Vietnam.

(b) Evaluation of seating length

The target of this study is to investigate whether plastic hinge is a real possibility for typical bridge located in a low moderate seismic zone or not. To account for the possibility of plastic hinging and the associated large displacement, 22TCN-272-05 requires that the beam seat length (N) shall be taken empirical displacement formula (equation (2)). This formula is an estimate of displacements that may be achieved only in the event of inelastic behavior. In this study, plastic hinge is assumed to occur at the pier column even though for Level 1 earthquake motion. For the typical straight structures modeled, this study indicates that plastic hinge at the column base is a real possibility for bridges located in a low to moderate seismic zone.

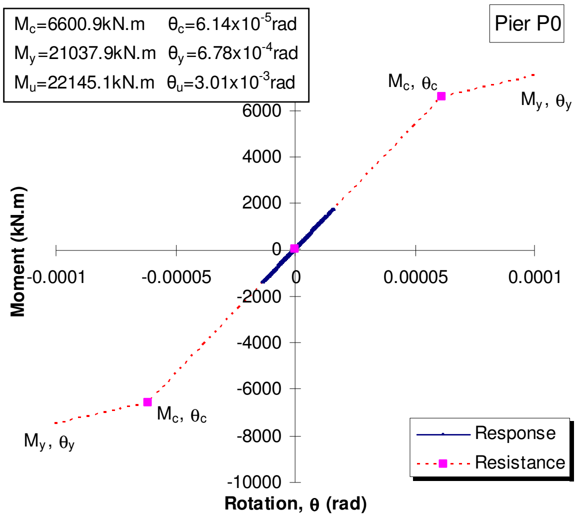

Fig. 9 Hysteretic response of the pier P0 at the plastic hinge

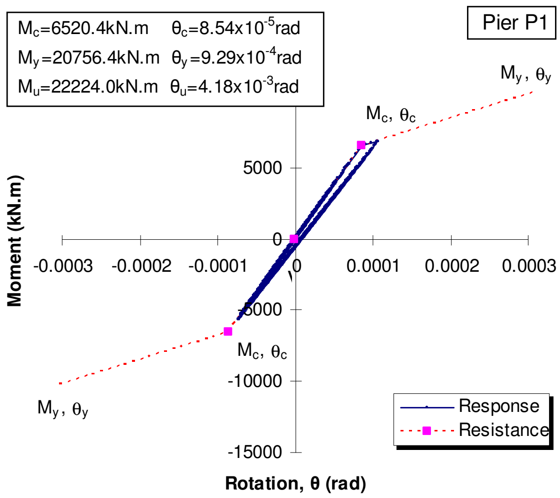

Fig. 10 Hysteretic response of the pier P1 at the plastic hinge

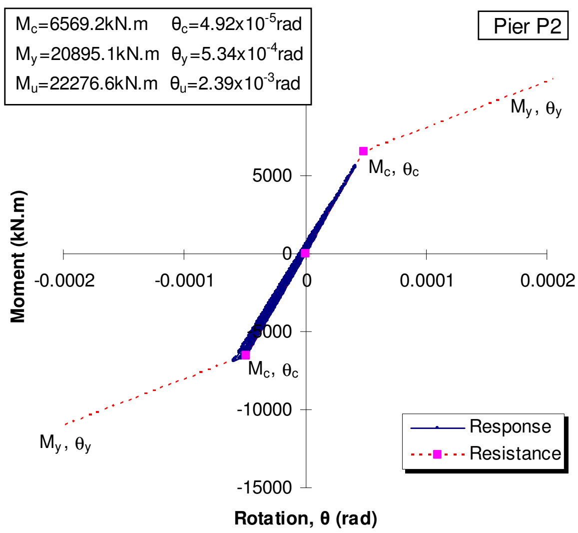

Fig. 11 Hysteretic response of the pier P2 at the plastic hinge

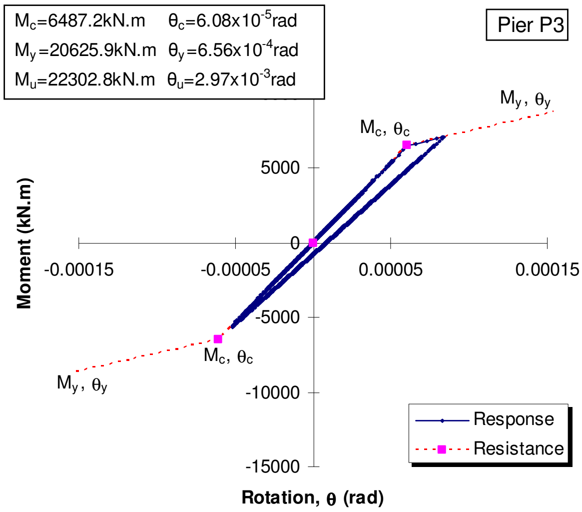

Fig. 12 Hysteretic response of the pier P3 at the plastic hinge

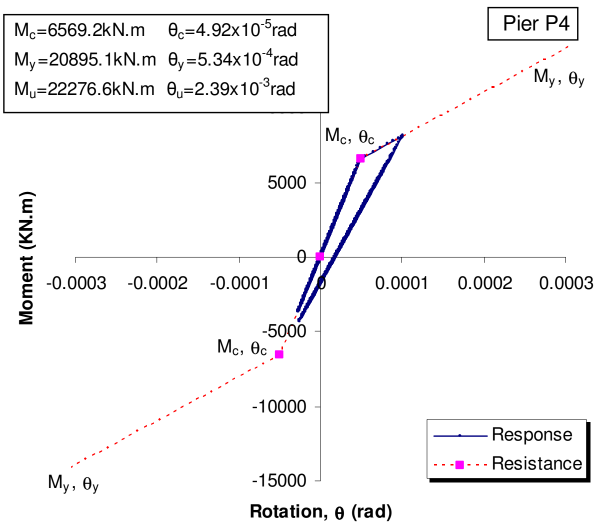

Fig. 13 Hysteretic response of the pier P4 at the plastic hinge

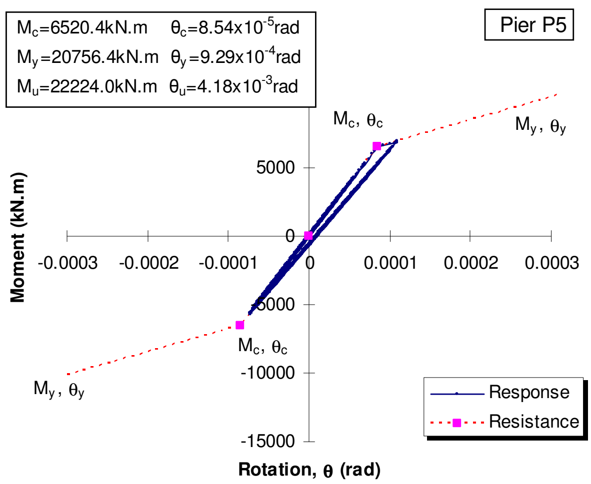

Fig. 14 Hysteretic response of the pier P5 at the plastic hinge

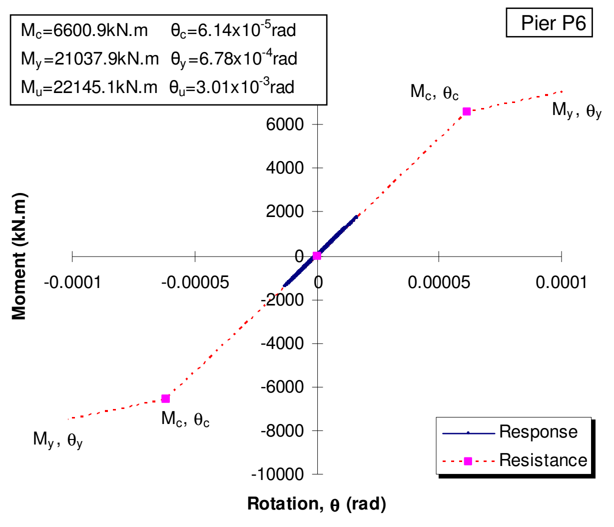

Fig. 15 Hysteretic response of the pier P6 at the plastic hinge

However, this seismic analysis with plastic hinge is not reasonable because it is an analytical model of typical bridge and earthquake ground motion is only 1.41 m/s^2. The empirical seat length designed at the abutment A1 for this typical bridge is N = 0.56 m with bridge modeled. The relative displacement between the substructure and the superstructure is defined by dynamic response analysis is 1.4 cm for level 1 in this study. Unseating of the girder will not be happened. The seat length N from 22TCN-272-05 has enough length to prevent the superstructure from departure and unseating.

5. CONCLUSIONS

The seismic design for bridge in Vietnam is review in this paper. Current Vietnam seismic design requires that an elastic analysis is performed to estimate design forces, and established a lower bound for design displacements. For the typical straight structures modeled, this study indicates that plastic hinge at the column base may occur in a low to moderate seismic zone such as Vietnam.

Although plastic hinge may occur, the stopper will restrict longitudinal deflections to values below those calculated by the empirical bearing seat formula.

For Level 1 earthquake motion according to Japan code, no serious damage is evaluated. The bearing capacity of the pier is still within elastic state. The bridge is also secured from the current earthquake occurring in Vietnam.

As the future research, more parametric studies and other earthquake wave should be conducted to verify real effect on bridge design in Vietnam. Since Vietnam specification was established based on AASHTO LRFD 1998, and classification of the seismic zones, earthquake motion etc., is different from America. Hence, to estimate seismic assessment of bridge in Vietnam is very important, then to modify some problems in the Specification in order to correspond with Vietnam conditions.

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