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Evaluation of seismic resistance for a multi-spans bridge in Vietnam by investigation of earthquake activity and dynamic response analysis

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The aseismicity of a multi-spans highway bridge designed by Vietnam Specification is evaluated by dynamic response analysis according to Japan Specification. The bridge adopted in this study is located at Hanoi in Vietnam where is classified to a low moderate seismic zone based on the investigation. As to Level 1 earthquake, there is no damage in the bridge, i.e. the seismic performance of the bridge is also secured by Japan Specification. As to Level 2 earthquake, though girder's unseating is not calculated, the bridge pier is predicted to have damage due to the large rotation angle at the plastic hinge and the lack of the shear resistance of the pier.

Keywords: bridge in Vietnam, earthquake activity, dynamic response analysis, stopper

1. INTRODUCTION

The arrangement of the social infrastructure is now rapidly progressing with the social advance and economic development in Vietnam. Recently, many bridges have been constructed and some bridges are still under constructing or planning. The bridge takes an important role in road and railway networks. Of particular note, the recent 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, caused serious damage to many lifeline facilities, including bridges. Vietnam has not experienced the big earthquake damages up to now and does not have the history of large scale earthquakes. The seismic designs for the bridges weren't adopted until former century 90's. Many seismometers are installed in Vietnam and some middle scale earthquakes have been recorded by the seismometers installed in Vietnam. According to the analyses of the earthquake records obtained by the seismometers, Vietnam is located at moderated seismic activity area. However seismic design for buildings and bridges become to be regarded as important design and so on in Vietnam. Seismic Design Specifications for Bridges in Vietnam (a part of 22TCN 272-05) was established based on AASHTO LRFD 1998 in 2001, officially applied in 2005. Recently many bridges are designed by this Specification, including some bridges constructed under Japanese financial support (ODA).

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, however, the seismic design based on Japan Specification must be carried out by static analysis for level 1 and level 2 earthquake, and dynamic analysis for level 2 earthquake, as to the seismic load used in the design is modified by the zone factor. In addition, it is uncontradictable that there is potentiality for large-scale earthquake to happen in Vietnam. As a study, it is necessary to verify the seismic performance of the bridges in Vietnam in case of a large-scale earthquake. To evaluate the aseismicity of the bridges in Vietnam and compare the difference between Vietnam Specifications for Bridge Design and Japan Specifications for Highway Bridge, the earthquake activity in Vietnam is firstly investigated and a multi-spans highway bridge designed by Vietnam Specification is adopted as an example to compare the results for these two design codes.

2. STATUS OF EARTHQUAKE AND SEISMIC DESIGN FOR BRIDGE IN VIETNAM

2.1 Earthquake activities in Vietnam

Vietnam is situated between two major faults: Himalayan Fault and Pacific Fault. These faults have strong effect on seismo-tectonic movement of this region. Moreover, Red River fault running through the north portion of Vietnam has caused several earthquakes in this region. According to the earthquake catalog compiled by the Vietnam Institute of Geophysics, 90% of the earthquakes took place in Vietnam is in Northwestern Vietnam as shown in **Fig. 1**. Aside from this region, there was no report of earthquakes of a magnitude larger than 5.5. Vietnam has been classified as a low seismicity region.

Some of the significant earthquakes happened recently in this region are presented in **Table 1**. The last two earthquakes caused strong shakings in Hanoi. Most recently, on 19 February, 2001, an earthquake of 5.3 Richter scale was occurred in Dienbien (close to the border with Laos PDR) at a depth of 12 kilometers. This earthquake was caused by the Laichau - Dienbien rupture, parallel with the Hong (Red) River rupture. Although the earthquakes happened up to now are moderate ones it is predicted that the Hong (Red) River rupture has potential of causing earthquakes of 7 - 8 Richter scale. In case those large scale earthquakes happen, the buildings and structures in this area of thousands square kilometers may be destructed [1].

In Vietnam, during the 20 Century 2 earthquakes with intensity $I_0 = 8 - 9$ (MSK-64) and magnitude $M = 6.5 - 7$ Richter degrees, 15 earthquakes with $I_0 = 7$ and $M = 5 - 5.9$, and more than 100 earthquakes with $I_0 = 6 - 7$ and $M = 4.5 - 4.9$ were observed according to "Research and Forecasting Earthquakes and Foundation Fluctuations in Vietnam" (Prof. Nguyen Dinh Xuyen and his collaborators, Vietnam Institute of Geophysics). As shown in **Table 2**, earthquakes of 5.5 - 6.8 on the Richter's scale would happen at 30 areas in Vietnam. In Hanoi capital, the tectonic fault exists along the Hong (Red) River, Chay River and Lo River. The seismographic survey shows that earthquakes of magnitude $M_S = 5.0$ have occurred in North Vietnam, in the north border of Hanoi depression and in the coastal zone of the central part of Vietnam. **Table 3** is a catalog of some of strong earthquakes in Hanoi region. The events from 1930 to the end of 2006 are all the instrumental data of the seismological network of the Northern Vietnam, China and ISC. Magnitude M_S used in **Table 3** is the surface wave magnitude; I_0 is seismic intensity in epicenter estimated following MSK scale that is similar to MM scale. The error of magnitude estimation is about 0.2 - 0.3 and of intensity I_0 is about 0.3 - 0.5 [2].

2.2 Seismic design for bridge in Vietnam Specifications

Vietnam Specifications of bridge design (referred to as 22TCN-272-05) published in 2005 was established according to AASHTO LRFD 1998. This specification also concerns with seismic design, some objects are modified according to Vietnam conditions such as the classification seismic zone, map of maximum seismic intensity zone, etc [3]. However, the concept of structure analysis and seismic analysis are originally taken from AASHTO LRFD 1998.

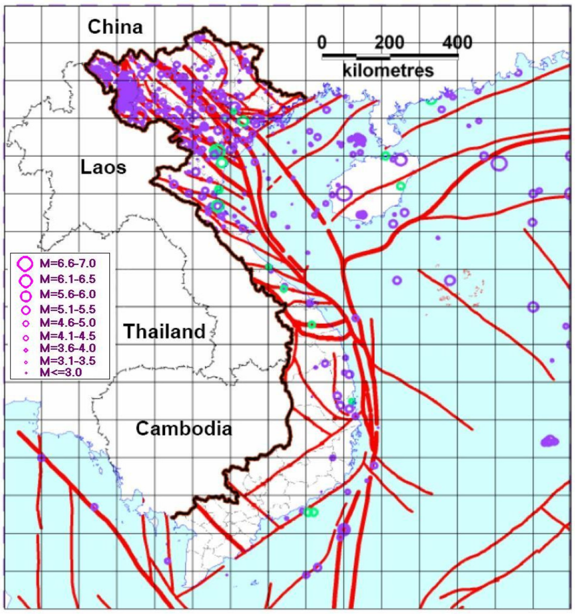


Fig. 1 Map of active faults and earthquake epicenters distribution in Vietnam from 1067 to 2002 (Nguyen, 2007).

Table 1 Catalog of strong earthquakes in Vietnam

No	Year	Location	Depth (km)	Magnitude M_S	Intensity I_0
1	1961	Bacgiang	28	5.5 - 5.9	7
2	1983	Tuangiao	13	5.3	7
3	1987	Nhanam	10	4.6	6
4	1988	Campha	10	4.9	7
5	2001	Dienbien	12	5.3	7

Table 2 List of areas in risk for strong earthquakes in Vietnam (Nguyen Dinh Xuyen et al., 2005)

No.	Area	Maximum earthquake (Richter)	No.	Area	Maximum earthquake (Richter)
1	Son La	6.8	16	Song Ma - Fumaytun	6.5
2	Dong Trieu	6.0	17	Red River - Chay River	6.0
3	Song Ca - Khe Bo	6.0	18	Rao Nay	5.5
4	Cao Bang-Tien Yen	5.5	19	Hanoi's Northeastern sunken area	5.5
5	Cam Pha	5.5	20	Lo River	5.5
6	Phong Tho - Than Uyen Muong La - Cho Bo	5.5	21	Da River	5.5
7	Muong Nhe	5.5	22	Ma River's lower section	5.5
8	Hieu River	5.5	23	Khe Giua - Vinh Linh	5.5
9	Tra Bong	5.5	24	Hue	5.5
10	Da Nang	5.5	25	Tam Ky - Phuoc Son	5.5
11	Poco River	5.5	26	Ba River	5.5
12	Ba To - Cung Son	5.5	27	109.5 meridian	5.5
13	Tuy Hoa - Cu Chi	5.5	28	Thuan Hai - Minh Hai	5.5
14	Vung Tau - Ton Le Sap	5.5	29	Hau River	5.5
15	Phu Quy 1	5.5	30	Phu Quy 2	5.5

Table 3 Catalog of some strong earthquakes near site construction (Hanoi, Vietnam) to the end of 2006 [2)]

No	Year	Depth (km)	Magnitude M_S	Intensity I_0	No	Year	Depth (km)	Magnitude M_S	Intensity I_0
1	1934	17.1	5.0	6	6	1961	28 ± 7	5.6 ± 0.4	7 ± 0.3
2	1939	12.8	4.7	6 ± 0.3	7	1967	20	5.0 ± 0.4	6 ± 0.3
3	1942	7 ± 1	4.4 ± 0.2	6 ± 0.3	8	1974	17 ± 13	4.1 ± 0.7	5 ± 0.3
4	1945	11	4.7 ± 0.5	6 ± 0.3	9	1989	5 - 6	4.9	7
5	1958	20 ± 5	5.3 ± 0.3	6 ± 0.3	10	2005	-	4.7	-

Table 4 Seismic zones in Vietnam

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

Table 5 Site coefficients (22TCN 272-05)

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

The seismic zone in Vietnam is classified into three zones as presented in **Table 4**. The map of acceleration coefficient in Vietnam is shown in **Fig. 2** [4)]. It is given by Vietnam Institute of Geophysics with return period about 500 years.

Earthquake loads are given by the product of the mass and the elastic seismic response coefficient C_{sm} for the m^{th} vibration mode. The coefficient is given as following equations.

$$C_{sm} = \frac{1.2AS}{T_m^{2/3}} \leq 2.5A \qquad (T_m \leq 4.0\text{ s}) \qquad (1a)$$

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

where T_m is the period of the m^{th} 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. 2**) and, it is 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 is the site coefficient as shown in **Table 5**. The four descriptive soil types are defined as follows [3)]:

+ 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.

+ 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. As for the important especial bridges such as Nhattan Bridge (Cable stayed bridge), etc, the seismic design is conducted based on such as Japan Specifications for Highway Bridge (referred to as JRA-2002) to secure more safety of the bridges.

3. CONSIDERING SOME CHARACTERISTICS IN SEISMIC DESIGN BETWEEN 22TCN-272-05 AND JRA-2002

In seismic design, it is thought that the peak value of design horizontal seismic coefficient is very important. Focusing on the determination method of this coefficient for moderate level earthquake, 22TCN-272-05 and JRA-2002 are compared. As to 22TCN-272-05, from equation (1a) and (1b), elastic seismic response coefficient can be presented as follows:

$$C_{sm} = f\,(A, S, T_m) \qquad (2)$$

The design horizontal seismic coefficient for level 1 earthquake according to JRA-2002 is as follows [5]:

$$k_h = C_z\, k_{h0} \qquad (3)$$

where k_{h0} = f (T_G, T) as presented in **Table 7**.

It is found from (2) and (3) that there is similarity in the seismic coefficient determined by the seismic zone, surface ground condition and the vibration characteristics of the bridge for both 22TCN-272-05 and JRA-2002:

- Seismic zone: modification factor C_z and acceleration coefficient A.

- Ground type: T_G (Type I, II, III) and site coefficient S = I, II, III, IV.

- Natural period: T and T_m

- Moderate earthquake Level 1 in Japan is similar to moderate earthquake in Vietnam.

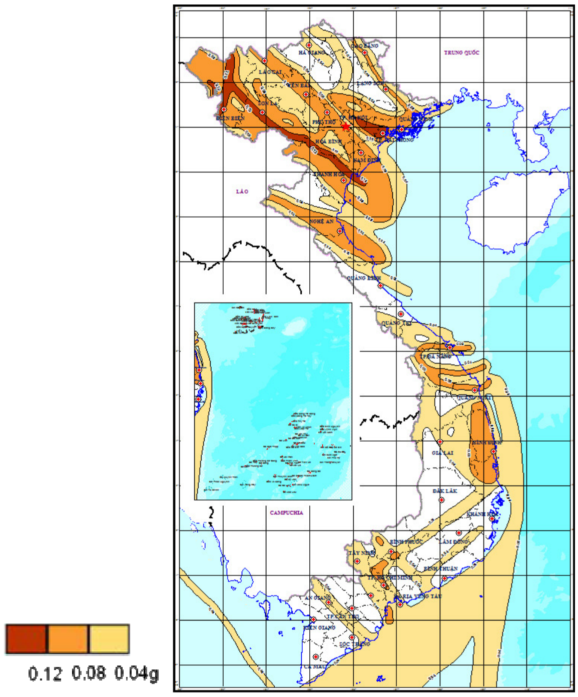


Fig. 2 Seismic zones and maximum seismic intensity zone of Vietnam with return period about 500 years [4]

The difference between these two codes is the values of the parameters. As to the PGA, 0.16 - 0.24 is adopted in JRA-2002, but 0.00 - 0.29 is used in 22TCN-272-05. As an example, some of the PGA values provided by Vietnam Institute of Geophysics for some construction sites are presented in **Table 6**. **Table 8** shows some values of input data characteristics in seismic design codes at both JRA-2002 and 22TCN-272-05.

Base on above observation, it is clear that some values of seismic design are similar but earthquake characteristics and seismic modification factors are different in these two codes. It is significant to evaluate the seismic resistance of bridge with two codes (JRA-2002 and 22TCN-272-05).

4. DESCRIPTION OF SEISMIC ANALYSIS PROCEDURES

4.1 Bridge structure

Fig. 3 shows the profile of a multi-spans continuous existing bridge in Vietnam. The superstructure is a hollow slab beam structure with 8 spans. The total length of the bridge is 250 m. The surface layer consists of medium sand, fine sand and gravelly sand as shown in **Fig. 4**. The thickness of the surface layer is about 40 m. The cross section of girder is shown in **Fig. 5**. The bridge consists of 3 rigid frame piers (P2, P3, and P4) and 4 bent piers. Rubber bearing supports (P0, P1, P5 and P6) are installed. The basic components of rubber bearing are elastomer and steel plates as shown in **Fig. 6**. The pier columns and the abutments are

Table 6 Acceleration coefficient used to seismic design of some bridges in Vietnam

No	Name of Bridge	Typical Bridge	Span Layout	MSK-64 Class	Acceleration coefficient (g)
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	Đa Bac	Cantilever Bridge	65+100+65	7	0.07
8	Non Nuoc	Cantilever Bridge	42+52+85+52+42	7(8)	0.10
9	My Thuan	Cable stayed Bridge	150+350+150	6	0.10
10	Ben Luc	Cantilever Bridge	50+90+120+90+50	7	0.10
11	Nhat Tan	Cable stayed Bridge	150+4@300+150	7(8)	0.12
12	Phu Long	Cantilever Bridge	75+120+75	7	0.08
13	Đong Tru	CFST arch bridge	80+120+80	8	0.17

Table 7 Characteristics of JRA-2002 with Level 1

Ground type	k_{h0}, value in term of natural period, T (s)	Characteristic value of ground, T_G (s)
Type I	$k_{h0} = 0.2$	$T_G < 0.2$
Type II	$k_{h0} = 0.25$	$0.2 \leq T_G < 0.6$
Type III	$k_{h0} = 0.3$	$0.6 \leq T_G$

Table 8 Seismic design conditions according to Japan and Vietnam specifications

Items	Japan Specifications for highway bridge (JRA-2002)	Vietnam Specifications for bridge design (22TCN-272-05)
Seismic zones	Modification factor: $C_z = 0.7$ (region C)	Acceleration coefficient: A = 0.1 g (region 2)
Ground characteristics	Dynamic characteristic value of the surface ground: $T_G = 0.82$ s > 0.6 s → Type III ground (soft ground or alluvial ground).	Site coefficient: Type III, S = 1.5 (Soft to medium stiff clay, sand, or other cohesion-less soil generally greater than 9m deep)
Damping ratio	Pier (NLE): 2%, pier (LE): 5%, girder (LE): 3%, bearing spring: 4%, footing: 10%	-
Seismic coefficient	Design horizontal seismic coefficient: $k_h = c_z k_{h0}$	Elastic seismic response coefficient: C_{sm}

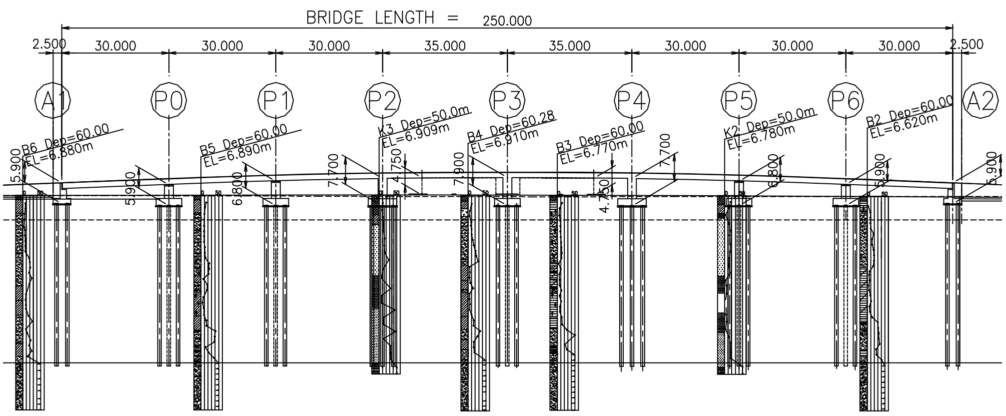


Fig. 3 Profile of a highway bridge (unit is mm)

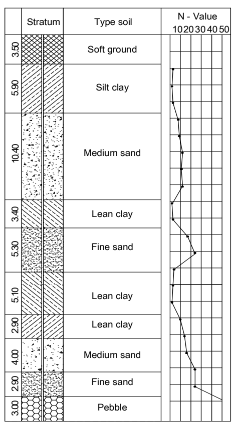


Fig. 4 Borehole log

-fabricated by reinforced concrete and bored cast-in-place piles are driven under the footing. The profile of the pier and the foundation is shown in **Fig. 7**.

4.2 Analytical modeling

The bridge used in this study is designed by static analysis according to the Vietnam design code. The stopper and rubber bearing are installed to enhance the seismic performance. According to the seismic design specified in Japan Specifications for Highway Bridge, a nonlinear analysis model of the entire bridge system as shown in **Fig. 8** is made to estimate seismic performance of the bridge, especially bearing capacity of the members and unseating of the girders. **Fig.9** shows detail model of the pier with the foundation, the stopper and the rubber bearing. The girders are replaced to linear beams.

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 $T_G = 0.82$ s > 0.60 s i.e. ground type in **Table 9** is type III ground. The modified factor C_z is selected as have 0.7 (region C). The concrete block as a stopper is installed at the top of the pier as shown in **Fig. 10**. The stopper is replaced to a spring element considering the spacing. The rubber bearing is replaced to a bi-linear spring in horizontal direction. The pier has circler cross section as shown in **Fig. 11**.

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 shown in **Fig. 12**. 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. The length of the plastic hinge is calculated by following equations and the results are presented in **Table 10**.

$$L_p = 0.2h - 0.1D \tag{4a}$$

$$\text{in which: } 0.1D \leqq L_p \leqq 0.5D \tag{4b}$$

where L_p is plastic hinge length; h is height of the column pier; D is section depth (D is diameter of a circular section). The Rayleigh damping coefficients are calculated from the vibration mode of the structure.

Table 9 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
T = 0.82 (s) > 0.60 (s) → Type III ground according to JRA-2002					

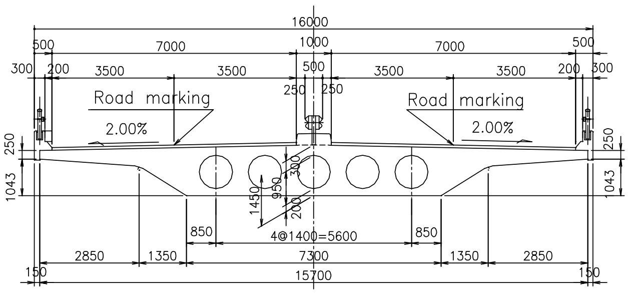


Fig. 5 Cross section of the girder (unit is mm)

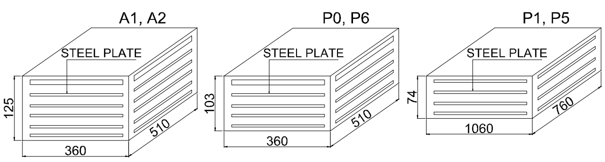


Fig.6 Dimensions of rubber bearing (unit is mm)

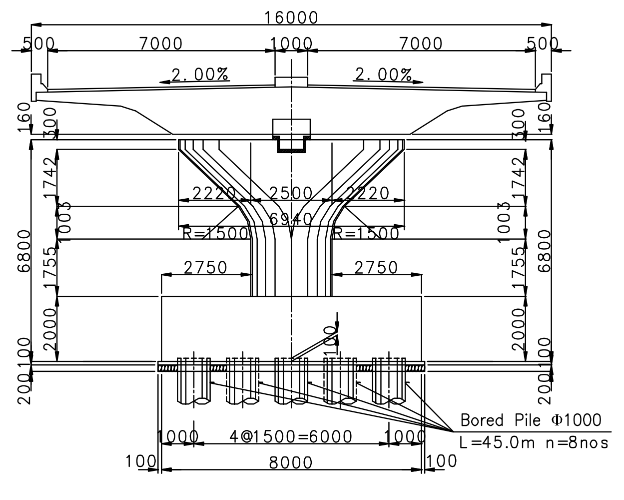


Fig.7 Profile of pier and foundation (unit is mm)

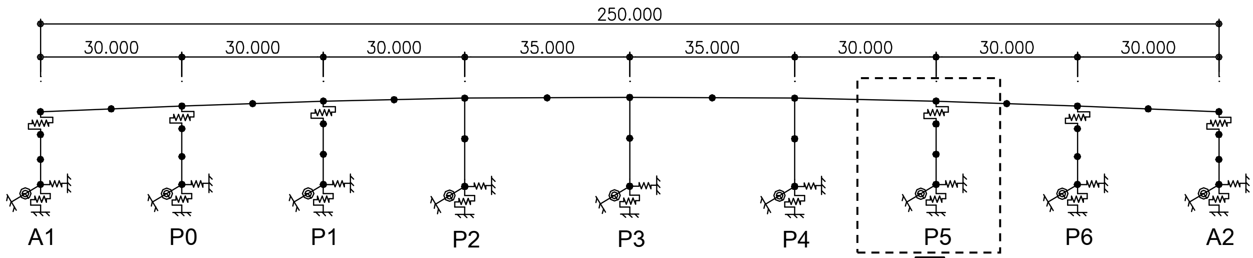


Fig. 8 Modeling of the entire bridge

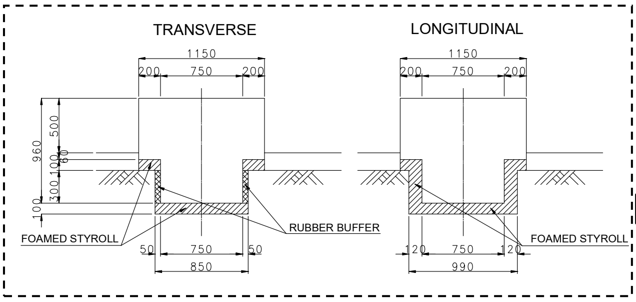


Fig. 10 Concrete stopper (unit is mm)

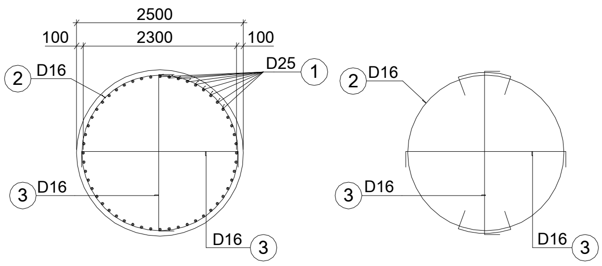


Fig. 11 Cross section of pier column (unit is mm)

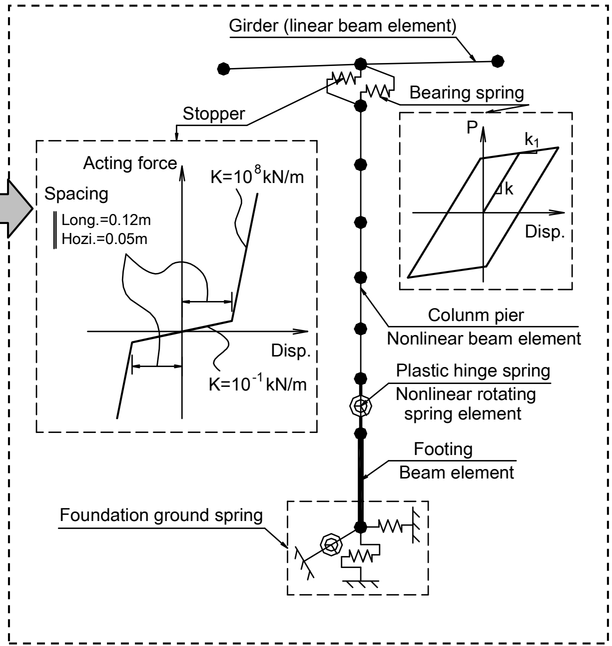
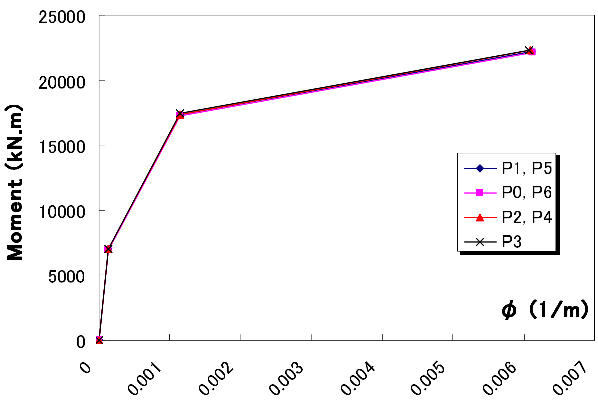
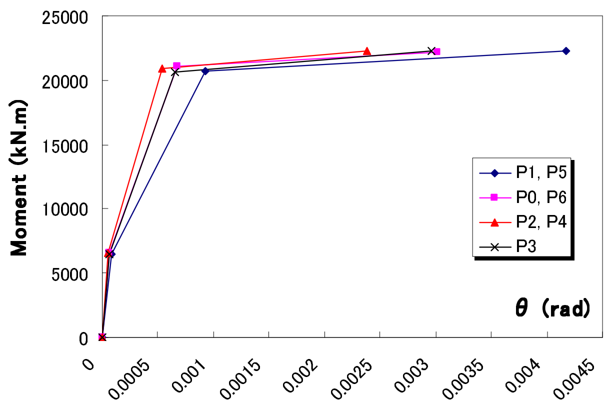


Fig. 9 Modeling of the bridge pier



a) Moment vs. curvature relation



b) Moment vs. rotation relation at the plastic hinge

Fig. 12 M - ϕ and M - θ relationships of column piers

Natural dominate frequencies of the structure are 2.337 H_z and 7.305 H_z. The values of the Rayleigh damping coefficients are $\alpha = 0.45829$ and $\beta = 0.00225$, 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 0.01 s. The damping ratios for members are presented in **Table 11**.

4.3 Input motions for dynamic response analysis

Two ground acceleration records in Japan are adopted as input

Table 10 Plastic hinge length of pier, L_p (m)

Pier	P0, P6	P1, P5	P2, P4(*)		P3(*)	
h (m)	3.9	4.8	5.7		5.9	
L_p m)	0.5	0.7	0.4	0.4	0.5	0.5

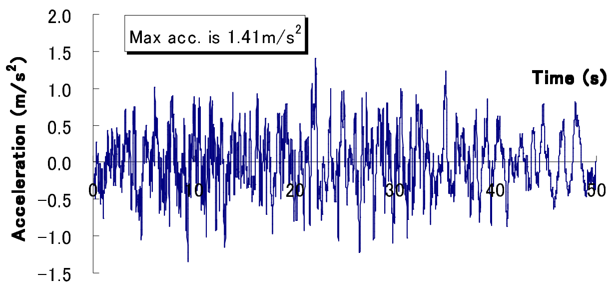
(*) The piers P2, P3, P4 have 2 locations of the plastic hinge length at bottom and top of pier

Table 11 Damping ratio of the members

Member	Damping ratio
Bridge column - pier (nonlinear member)	2%
Bridge column - pier, footing (linear member)	5%
Girder (linear member)	3%
Bearing spring	4%
Foundation spring	10%

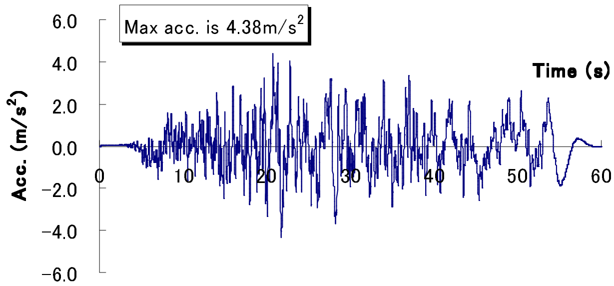
Table 12 Intensities and PGA at site construction [2)]

No	Return Periods (years)	Site construction A (cm/s^2)	Site construction I_0 (MSK)
1	200	67.38	6
2	500	85.56	6 - 7
3	1000	100.26	7
4	2000	116.53	7
5	5000	138.24	8
6	10000	155.84	8



a) Horizontal acc. in the Tsugaru Ohhashi, 1983

(M_g = 7.7, the maximum acc. is 1.41 m/s^2)



b) Horizontal acc. in the Kushirogawa, 1994

(M_g = 8.2, the maximum acc. is 4.38 m/s^2)

Fig. 13 The ground acceleration records

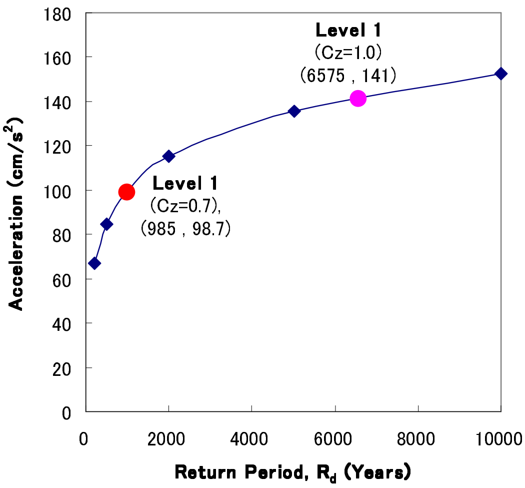


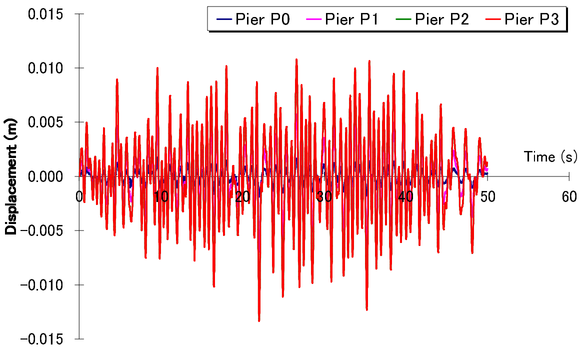
Fig. 14 PGA with return periods at the near construction site in Hanoi, Vietnam

-data at the ground level. Namely, Tsugaru Ohhashi (1983) and Kushirogawa (1994) as shown in **Fig. 13** are adopted here. The peak ground accelerations of them are 1.41 m/s^2 and 4.38 m/s^2, respectively. These peak ground accelerations are corresponded to Level 1 ground motion and Level 2 ground motion according to Seismic design for bridge in Japan Specifications (JRA-2002), respectively. These ground acceleration records are adopted because the soil condition of the construction site is classified into Group III in soil condition.

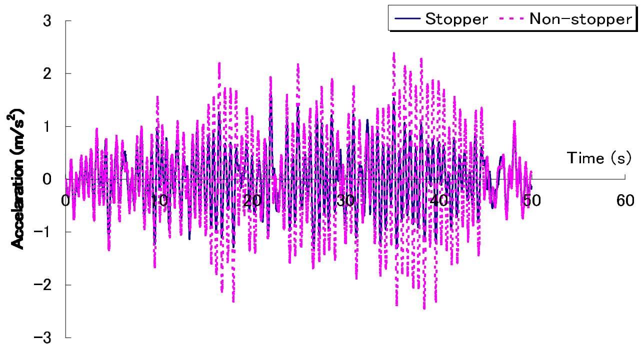
Table 12 shows the intensity (MSK-64 scale) and PGA with different return periods at the construction site of this bridge. This table is calculated from earthquake events data as presented in **Table 2** and **Table 3**.

Fig. 14 shows relationship between the PGA (cm/s^2) and return periods at the near construction site of the bridge. For Level 1 earthquake motion, modified factor C_z = 0.7 is adopted because this area is located at moderate earthquake activity. Only Level 1 earthquake is presumed to occur and Level 2 is not presumed to occur in this region. However, return period of Level 1 is much more at site construction yet and return period is 985 year for C_z = 0.7 and 6575 year for C_z = 1.0, respectively. While return period of level 1 (JRA-2002) is about 50 to 100 years i.e., there is large difference as to return periods of Level 1 earthquake motion between Vietnam and Japan. For Level 2 earthquake, its return period is extremely long.

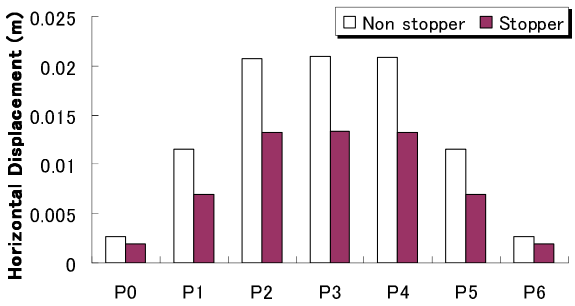
From view point of earthquake activities in Hanoi, Level 2 earthquake motion may not be considered in the design work. However the damage of this bridge for Level 2 earthquake is studied here to confirm the damage level. It is maybe important to evaluate for inland earthquake about M_g = 6.5 - 7.0 as Level 2 earthquake motion in the future.



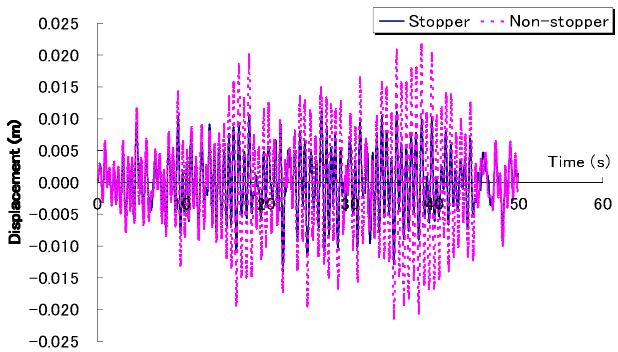
a) Displacement vs. time relation at the top of the piers



a) Acceleration of the girder end (A1)



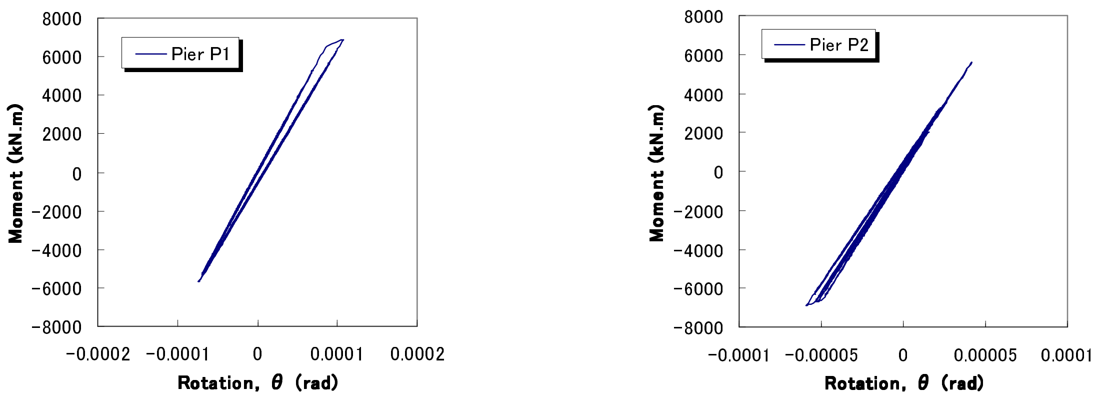
b) Considering displacement at the top of the piers for case of stopper and non-stopper



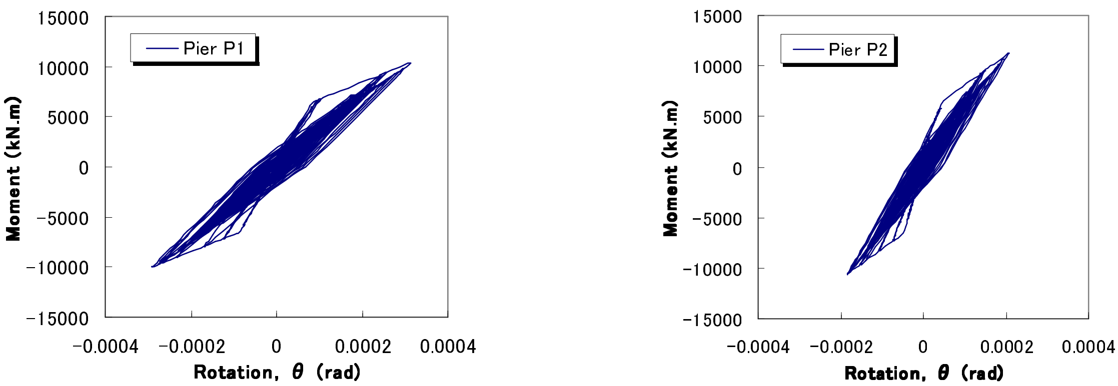
b) Displacement of the girder end (A1)

Fig.15 Displacement at the top of the piers for Level 1

Fig. 16 Response of the bridge for Level 1



a) Case of stopper



b) Case of non-stopper

Fig. 17 Hysteretic response at the plastic hinge for Level 1

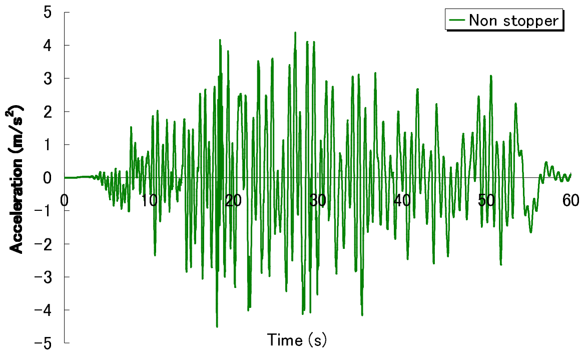
5. RESPONSE OF BRIDGE STRUCTURE IN A LOW-MODERATE SEISMIC ZONE (LEVEL 1)

Following to 22TCN-272-05, longitudinal stoppers shall be designed for a horizontal force calculated by the acceleration coefficient and the self weight of the girder. Sufficient slack shall be allowed in the stopper so that the restrainer does not start to act until the design displacement is exceeded. The small displacements of girder at pier P2, P3 and P4 are calculated as shown in **Fig. 15**. **Fig. 16** shows the response of the bridge for PGA of the input wave is 1.41 m/s^2. Results are compared as to existing of the stopper. The maximum accelerations of the girder are 1.62 m/s^2 and 2.47 m/s^2 while the maximum displacements of the girder are 1.4 cm and 2.2 cm for case of stopper and non-stopper, respectively. **Fig.** 17 shows the hysteretic response at the plastic hinge of the pier P1 that rubber bearing is installed at top and pier P2 that is rigid jointed between the pier and the girder. The maximum rotation angles of P1 and P2 are 1.072×10^{-4} rad, 4.149×10^{-5} rad and 3.134×10^{-4} rad, 2.062×10^{-4} rad for case of stopper and non-stopper, respectively. The displacement at the top of the pier is also small regardless of existing of the stopper. The results show that no damage is calculated under a moderate ground excitation In this case, the stopper plays a not significant role for the damage of the bridge.

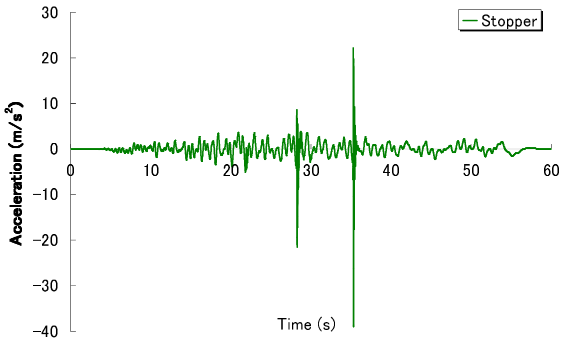
6. RESPONSE OF THE BRIDGE WITH STRONG GROUND EXCITATION (LEVEL 2)

Fig. 18, **Fig. 19**, **Fig. 20** and **Fig. 21** show the response of the girder when the peak ground acceleration is 4.38 m/s^2. The maximum accelerations of the girder are 38.6 m/s^2 and 4.52 m/s^2 while the maximum displacements of the girder are 10.13cm and 23.91 cm for case of stopper and non-stopper, respectively. The acceleration of the girder in case of the stopper is generally small due to the spacing but very large acceleration is generated when the girder movement stops suddenly due to the collision. Longitudinal displacement of the girder can be restrained by the stopper. When the stopper is not installed, fairly large displacement of about 24 cm is calculated.

Fig. 22 shows the hysteretic response at the plastic hinge of the pier P1 (rubber bearing) and pier P2 (rigid pier and girder). The maximum rotation angles of P1, P2 are 3.15×10^{-2} rad, 3.67×10^{-2} rad and 4.06×10^{-2} rad, 8.34×10^{-2} rad for case of stopper and non-stopper, respectively. The displacements at the top of the pier are large with both cases. The maximum displacements at the top of pier are 10.13 cm and 23.91 cm for case of stopper and non-stopper, respectively. Almost piers have damage and the residual displacement limit of the pier $\delta/h \leq 1/100$ is not satisfied. Where δ is displacement at top of the pier, h is height of the pier.



a) Case of non-stopper



b) Case of stopper

Fig.18 Acceleration of the girder end (at A1)

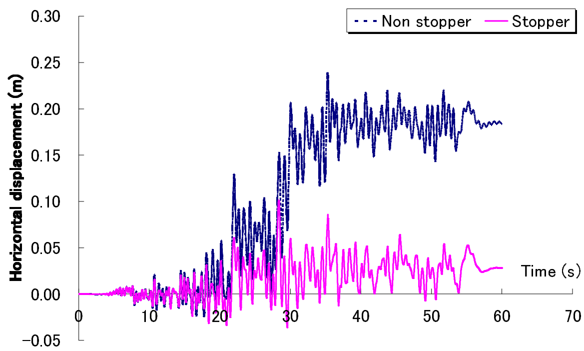


Fig. 19 Response of the girder for Level 2

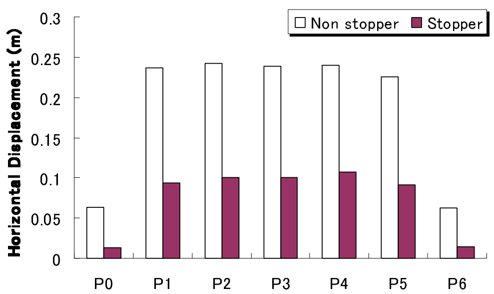
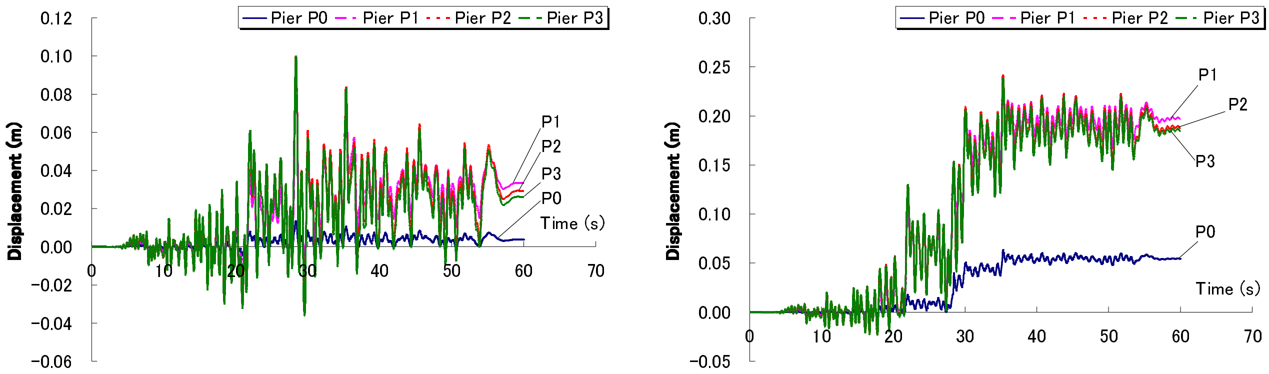
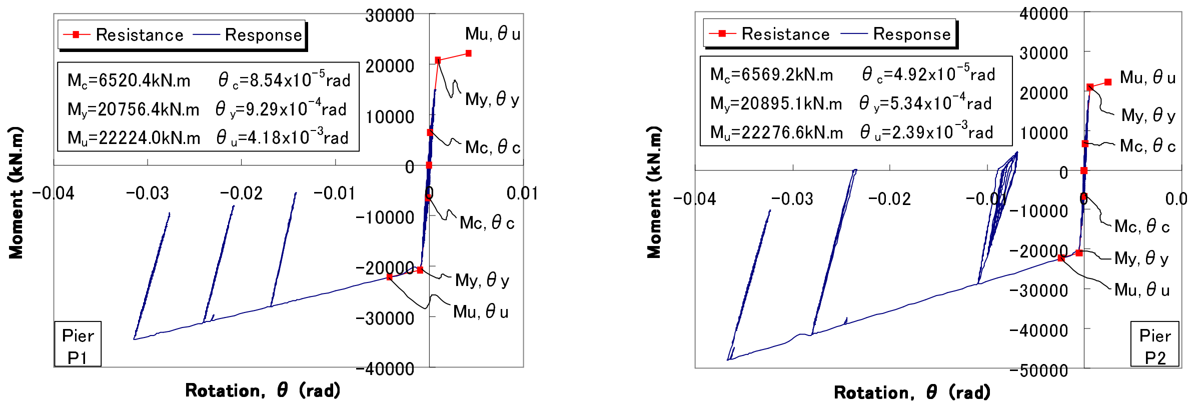


Fig. 20 Displacement at the top of the piers for Level 2

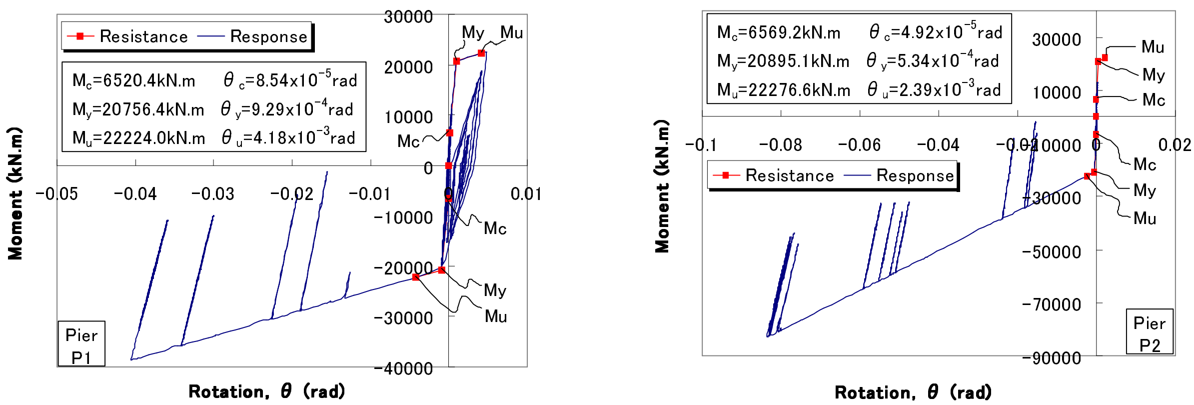


a) Case of stopper b) Case of non-stopper

Fig. 21 Displacement at top of pier for Level 2



a) Case of stopper



b) Case of non-stopper

Fig. 22 Hysteretic response at the plastic hinge for Level 2

Table 13 Summary of the results at P1 pier

Items	JRA-2002 (Response)			22TCN-272-05 (Resistance)
	Level 1		Level 2	
	$C_z = 0.7$	$C_z = 1.0$	$C_z = 0.7$	
Bending moment (kN·m)	6914	9423	21250	18538
Shear resistance (kN)	1588	2163	4841	3276
Displacement of pier (m)	0.007	0.015	0.094	0.048

7. EARTHQUAKE EFFECTS TO EXISTING BRIDGE

7.1 Shear and moment resistance of the pier

For shear resistance capacity of the pier, both JRA-2002 and 22TCN-272-05 are provided by shear strength of a reinforced concrete pier which consists of the concrete section and the spiral section. In 22TCN-272-05, the shear resistance capacity (V_r) is calculated by the equation of $V_r \le \varphi V_n$, where $\varphi = 0.85$ is resistance factor. The nominal shear resistance V_n shall be determined as the lesser of bellow two equations.

$$V_n = V_c + V_s \quad (5a)$$

$$V_n = 0.25\ f'_c\, b_v\, d_v \quad (5b)$$

in which:

$$V_c = 0.083\beta\ \sqrt{f'_c}\, b_v d_v \quad (5c)$$

$$V_s = \frac{A_v f_y d_v (\cot g\theta + \cot g\alpha)\sin\alpha}{s} \quad (5d)$$

where b_v is effective web width (mm), d_v is effective shear depth (mm), s is spacing of stirrups (mm), β is factor indicating ability of diagonally cracked concrete to transmit tension, θ is angle of inclination of diagonal compressive stresses (deg), α is angle of inclination of transverse reinforcement to longitudinal axis (deg), A_v is area of shear reinforcement within a distances (mm^2). The flexural resistance capacity (M_r) is calculated by the equation of $M_r \le \varphi{\cdot}M_n$, where $\varphi = 0.85$ is resistance factor.

Take the typical pier P1 as an example to observe, besides for extreme capacity of pier, the case of modified factor $C_z = 1.0$ is also considered. The results shows the pier has no damage to low moderate earthquake level 1 in both case of modified factor $C_z = 0.7$ and $C_z = 1.0$ as shown in **Table 13**. However the pier has severe damage by strong earthquake level 2.

If bearing capacity of the pier is satisfied for Japanese code, longitudinal reinforcement of D25 is replaced to D32 for bending moment and spiral D16 with 300 mm spacing is replaced to D19 with 150 mm spacing.

7.2 Evaluation of seating length

The seat length required to prevent unseating is considered. The seat length S_E (m) is determined from the Japanese Design Specifications for Highway Bridges as follows:

$$S_E = u_R + u_G \ge S_{EM} \quad (6a)$$

$$S_{EM} = 0.7 + 0.005 \times l \quad (6b)$$

$$u_G = \varepsilon_G \times L \quad (6c)$$

where u_R is the relative displacement developed between a substructure and a superstructure (m), u_G is relative displacement of the ground along the longitudinal direction of a bridge (m), S_{EM} is a minimum seat length (m), ε_G is ground strain induced during an earthquake along the longitudinal direction of the bridge (equal to 0.005 for groups III ground conditions), L is a distance contributing to the relative displacement of the ground (m), and l is a span length (m). The seat length (JRA-2002) designed at the abutment A1 is:

$S_E = 0.24 + 0.005 \times 125 = 0.865\ \text{m} \ge S_{EM} = 0.7 + 0.005 \times 30 = 0.85\ \text{m}$.

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 as follows:.

$$N = (200 + 0.0017L + 0.0067H)(1 + 0.000125S^2) \quad (7)$$

where N is minimum support length measured normal to the centerline of bearing (mm); 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-span bridges (mm); S is skew of support measured from line normal to span (deg).

The empirical seat length (22TCN-272-05) designed at the abutment A1 is N = 0.88 m.

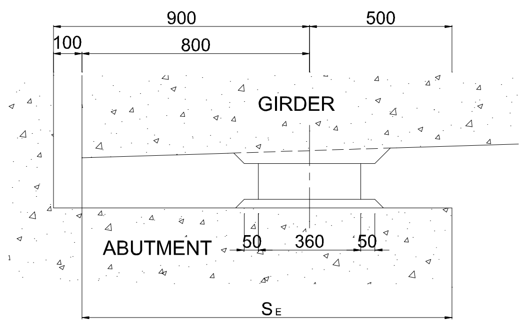


Fig. 23 Girder end support (mm)

The relative displacement between the substructure and the superstructure is defined by dynamic response analysis is 21.5 cm for level 2. This value is within both design values. Unseating of the girder will not be happened. **Fig. 23** shows the detail of the girder end. The seat length S_E from JRA-2002 and N from 22TCN-272-05 have enough length to prevent the superstructure from departure and unseating.

8. CONCLUSIONS

The earthquake events in Vietnam are investigated and, the seismic performance of a multi-spans highway bridge near to Hanoi designed by Vietnam Specification is verified by Japan Specification. Following conclusions can be obtained.

1) For Level 1 earthquake motion, no serious damage is evaluated. As to the calculation results, there are no distinct difference between Japanese code and Vietnam code.
2) For Level 2 earthquake motion, the bearing capacity of the pier as to shearing force and bending moment is un-sufficient. However level 2 earthquake motion that is predicted in Japan is hardly happened according to investigation of earthquake records near Hanoi.
3) When Japanese code as to Level 2 is applied to design for this bridge, retrofit of the pier is necessary. It is required that D25 is replaced to D32 for longitudinal direction reinforcement and D16 with 300 mm spacing is replaced to D19 with 150 mm for spiral reinforcement.
4) Relative displacement between the superstructure and the substructure for Level 2 is calculated by the dynamic response analysis. Seat length of the pier is about 80 cm for both design codes. The calculated values are within the designed seat length. The use of stopper may decrease relative displacement and prevent the girders from unseating when the pier is not collapsed.

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