COMPARISON OF SEISMIC RESISTANCE OF HIGHWAY BRIDGE IN YUNNAN BY THE SPECIFICATION OF CHINA AND JAPAN

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Yunnan province is one of the populated areas in China where earthquakes have frequently happened especially in recent years, meanwhile a large number of highway bridges have been designed and constructed in Yunnan, whose seismic resistance design is implemented according to the <Specifications of Earthquake Resistant Design for Highway Engineering>(JTJ004-89). Since October 1st 2008 the <Guidelines for Seismic Design of Highway Bridge> (JTG/T B02-01-2008) has been released, that has radically revised the guiding concept and performance requirement for the bridge seismic design. Then what level the bridges that have been designed and constructed before the date line are at according to the revised specification, what level according to the current <Specification for Japanese Highway Bridges> that is considered more advanced in the world, and what are the differences between the Chinese specification and the Japanese one are studied. In this paper, a representative expressway bridge in Yunnan respectively according to the Chinese old, revised specifications and Japanese specification is analyzed. This calculation brings some helps to the construction of the highway bridge in Yunnan.

Key Word: *Bridge in Yunnan, earthquake activity, nonlinear dynamic analysis, response spectrum, seismic specification for highway bridges*

1. INTRODUCTION

Yunnan province lies in the southwest of China, border Burma, Laos and Vietnam at south, as shown in **Fig.1.** and **Fig.2.**.

Yunnan province is one of the concentrative areas in China where earthquakes have frequently happened. In recorded history, magnitude 8 and 7-7.9 earthquakes happened 9 times and 78 times respectively. In the 46 years from 1950 to 1995 over magnitude 5 earthquakes totally happened 202 times, averagely 5 times per year, among them over magnitude 6 ones were 41 times and over magnitude 7 were 6 times, the frequency is highest in China. It can be seen in **Fig.3.** and **Fig.4.**. The **Table 1.** and **Table 2.** list the earthquakes happened in modern times in Yunnan. Frequent earthquakes are considered caused by the geographical position: Yunnan lies in the

Fig.1 Yunnan location in China

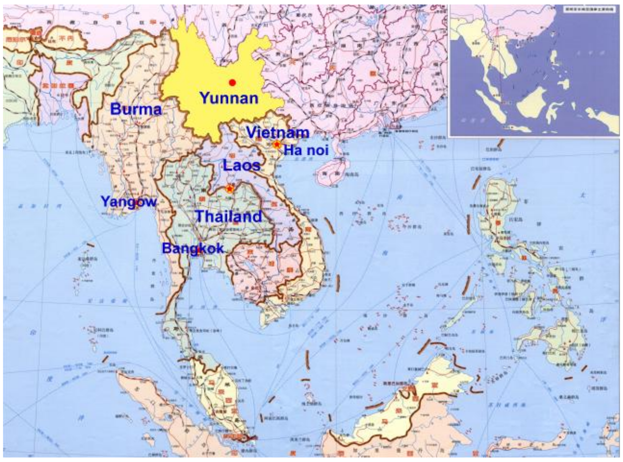

Fig.2. Yunan location in Southeast Asian

Fig. 3. Distribution of main seismic belts in China

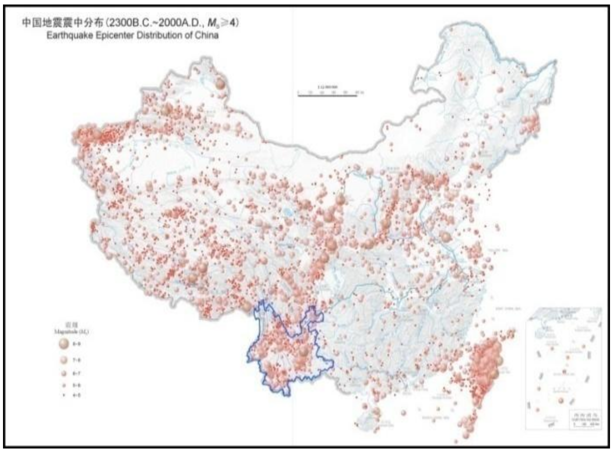

Fig. 4. 2300B.C.—2000A.D. epicenter distribution of over magnitude 4 earthquakes in China

southwest seismic belt that is the primary seismic belt of China, and also lies in the Himalayas-Mediterranean seismic belt that is one of the two main seismic belts in the world, whose diastrophism is so severe on the globe under the double effects of the subduction by Indian plate towards to east and the lateral extrusion of Qinghai-Tibet block.

Table 1. Catalog of over magnitude 6 earthquakes in 1949-2007 in Yunnan

No	Time	Location	Magnitude M_s
1	1951.12.21	Jianchuan, Heqing	6.3
2	1966.2.5	Dongchuan	6.5
3	1970.1.5	Tonghai,Eshan,Jianshui, Yuxi,Shiping	7.7
4	1974.5.11	Daguan, Yongshan	7.1
5	1976.5.29	Longlin, Luxi	7.4
6	1985.4.18	Luquan, Xundian	6.3
7	1988.11.6	Cangyuan, Genma	7.6
8	1995.6.30	Menglian	7.3
9	1995.10.24	Wuding	6.5
10	1996.2.3	Lijiang	7.0
11	1998.11.19	Ninglang	6.2
12	2000.1.15	Yaoan	6.5
13	2001.4.12	Shidian	5.9
14	2001.10.27	Yongsheng	6.0
15	2003.7.21	Dayao	6.2
16	2004.8.10	Nudian	5.6
17	2006.7.22	Yanjing	5.1
18	2007.6.3	Ninger	6.4

Table 2. Catalog of earthquakes in 2008 in Yunnan

No.	Time	Location	Magnitude M_s
1	2008.8.21	Yingjiang	5.9
2	2008.8.30	Yongren,Panzhihua	6.1
3	2008.10.8	Yuanmou	4.5
4	2008.12.26	Ruili	4.9
5	2008.12.26	Yiliang (Kunming)	4.3

The main secondary seismic belts in Yunnan are: (1) Mabian-Daguan; (2) Xiaojiang; (3) Tonghai-Shipin; (4) Tengcong-Longlin; (5) Genma-Lancang; (6) Simao-Puer. The earthquakes happened in these secondary seismic belts are characterized by high frequency and intensity, shallow seismic source and widely distribution. Furthermore in recent years there are more and more active crustal movement tendency discovered and several times severely destructive earthquakes already happened in the area. The epicenter of Wencuan earthquake in May 12nd, 2008 lies less than 600 km to Yunnan.

On the other hand, Yunnan has tremendously developed its highway infrastructure in recent 15 years under the National Strategy of Developing the Western Region. By the year of 2008, the total length of highway has reached 198.5 thousand kilometers, among them 2500 kilometers is expressway. **Fig.5.** is the distribution of the backbone highways in Yunnan. Among them the bridge and tunnel have more share along highway in Yunnan for its mountainous terrain. For example its proportion has reached 47.9% in Shuifu~Maliuwan expressway in the northwest of Yunnan. As an important component of highway net, Bridge plays momentous function in social and economic development, also plays a life-and-death role in seismic relief and reconstruction. However bridge also is the anti-earthquake weakness among various highway infrastructures for its structural characteristic. So it is considered necessary that the more attention is paid to improve the seismic resistance of bridges in Yunnan and guarantee their safety.

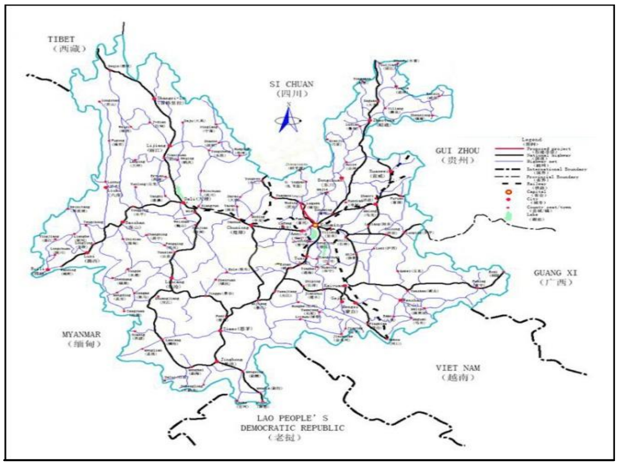

Fig.5. Distribution of highways in Yunnan

2. THE ESSENTIAL OF CHINESE SEISMIC SPECIFICATION

During the period from Jan. 1st 1990 to Set. 30th 2008 the bridge seismic design was carried out in China according to the <Specification of Earthquake Resistant Design for Highway Engineering>JTJ044-089 (the <China JTJ044-089> is abbreviated in the following) that was concluded from Tangshan Earthquake happened in Jul. 28th 1976. From Oct. 1st 2008 the revised vision called <Guidelines for Seismic Design of Highway Bridge> JTG/T B02-01-2008(the <China JTG/T B02-01-2008> is abbreviated in the following) was published as a recommended occupation standard. In some level the revised version embodies many newest seismic theories in the world. The essentials of the two specifications are summarized as the following.

(1)The <China JTJ044-089>

The earthquake force according to the provisions concerned is defined as the following equation：

$$E_{ihp} = C_i C_z K_h \beta_i \gamma_i X_{ii} G_i \tag{1}$$

Where E_{ihp} is horizontal earthquake force (kN) acting upon the No. i node of the column that has been discretized by the finite element method; C_i is the importance modified coefficient; K_h is the horizontal earthquake coefficient (the areal coefficient); γ_i is the participation coefficient of the fundamental mode; X_{ii} is the relative horizontal displacement of the center of gravity of the No. i element of column; G_i is the gravity of the No. i element of column; β_i is the dynamic modified coefficient of the fundamental period and is shown in **Fig.6.**; T is the fundamental period of the construction; C_z is the general influence coefficient and is listed in **Table 4.**

In addition, the design frequency is considered as shown in **Table 3.**

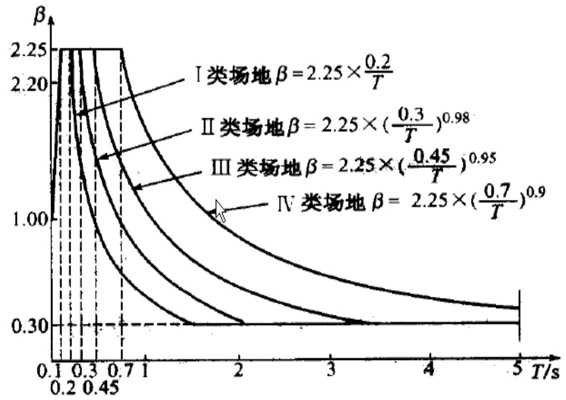

Fig. 6. Dynamic modified coefficient

Table 3. Design periods (years)

Importance modified coefficient (C_i)	General influence coefficient (C_Z)				
	0.20	0.25	0.30	0.33	0.35
1.7	50	75	106	129	147
1.3	31	46	64	76	85
1.0	21	29	40	47	52
0.6	10	13	17	20	22

Table 4. General influence coefficient (C_Z)

Classification by the bridge type, column and abutment			Height of column: H (m)		
			$H < 10m$	$10 \le H < 20$	$20 \le H < 30$
Beam bridge	Flexible pier	Column pier、pile bent pier、thin walled pier	0.30	0.33	0.35
	Gravity pier	Gravity pier that lies on the natural foundation or the open caisson foundation	0.20	0.25	0.30
	Column that lies on the multiple row piles		0.25	0.30	0.35
	Abutment		0.35		
Arch bridge			0.35		

Table 5. Performance level in the <China JTJ044-089>

Project category	Performance level
The works of Expressway and Class I Highway that lie on the areas with common geology condition	Can be normally used after be commonly repaired
The works of Expressway and Class I Highway that lie on the areas with soft cohesive soil layers or liquefied soil layers The works of Class II Highway that lie on the areas with common geology condition	Capable of recovering functions by emergency repair works within a short period
The works of Class III or Class IV Highway The works of Class II Highway that lie on the areas with soft cohesive soil layers or liquefied soil layers The works of Expressway, Class I and Class II Highway that lie on the faulted zones and their vicinity with high-risk geologic hazard	Bridges, tunnels and other important constructions can avoid serious damage

Table 6. Performance level in the <China JTG/T B02-01-2008>

Performance level	Performance Purpose	
	E1 effect	E2 effect
A Class	No damage or can be used without repair	Only insignificant damage and can be used after easy repair or without repair
B Class	Same as above	To sure no collapse or serious damage and is capable of recovering functions to meet emergency by temporary reinforce
C Class	Same as above	
D Class	Same as above	

(2)The <China JTG/T B02-01-2008>

The performance level in the specification is shown in **Table 6.**

As far as the analysis method, there is only exercisable and definite provisions and explication on the respond spectrum method to calculate earthquake force upon bridge, while by the time-history method and the power spectrum method there are only some macroscopic principles refered.

When the structural dumping ratio is 0.05, the response spectrum of the horizontal design acceleration is definite as **Fig 7**.

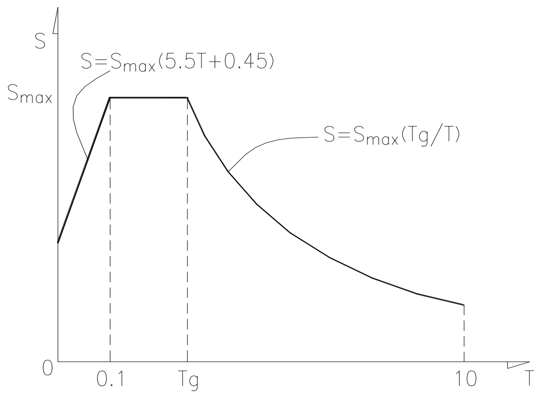

Fig. 7. Respond spectrum of the horizontal design acceleration

Where (a) S_{max} is the maximum value of the horizontal design acceleration and it is given as the following equation.

$$S_{max} = 2.25 C_i\, C_s\, C_d\, A \qquad (2)$$

In equation (2), C_i is the importance modified coefficient, C_s is the site coefficient as shown in **Table 7.** C_d is the dumping modified coefficient; A is the peak value of the earthquake acceleration with the corresponding design intensity as shown in **Fig.8.**

(b) T_g is the site characteristic period listed in **Table 8.**, in that the representational value is defined in **Fig.9.** by the location in this map.

(c) T is the natural period of the structure vibration.

The design period of the specification is considered as shown in **Table 9.**

Table 7. Site coefficients (C_s)

Site category	Basic intensity					
	6	7		8		9
	0.05g	0.1g	0.15g	0.2g	0.3g	0.4g
I	1.2	1.0	0.9	0.9	0.9	0.9
II	1.0	1.0	1.0	1.0	1.0	1.0
III	1.1	1.3	1.2	1.2	1.0	1.0
IV	1.2	1.4	1.3	1.3	1.0	0.9

Table 8. Modified value of characteristic period (s)

Representational value of the Characteristic period in zoning map (s)	Site category			
	I	II	II	IV
0.35	0.25	0.35	0.45	0.65
0.40	0.30	0.40	0.55	0.75
0.45	0.35	0.45	0.65	0.90

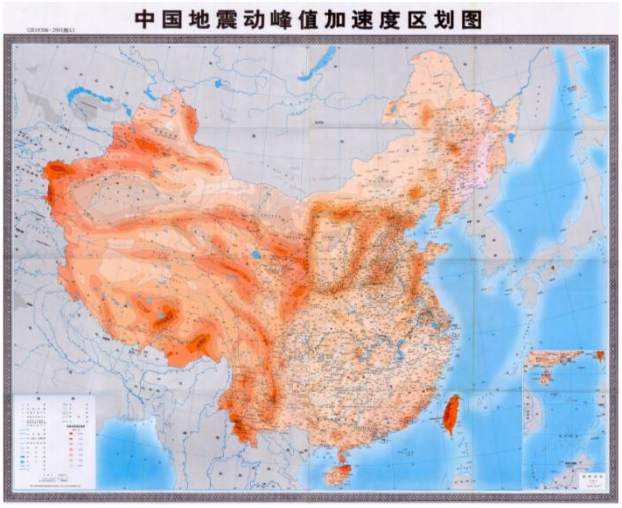

Fig. 8. China earthquake peak value acceleration zoning map

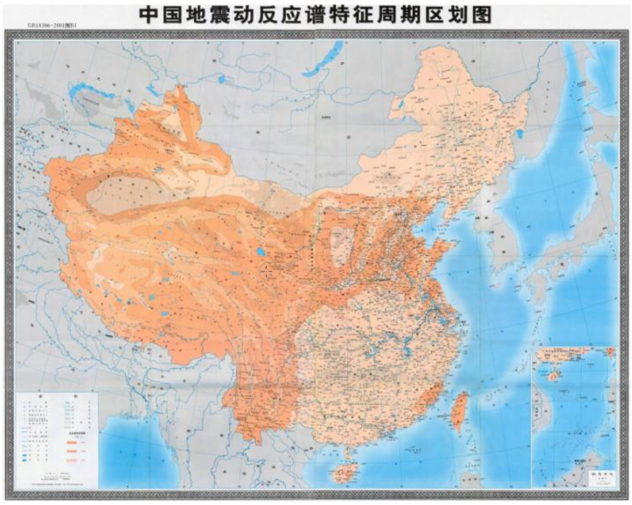

Fig.9. China earthquake characteristic period of response spectrum zoning map

3．COMPARISON OF PARAMETERS IN SEISMIC SPECIFICATIONS

The <Specification for Highway Bridges, Part V Seismic Design> (the <Japan JRA-2002> is abbreviated in the following) is implemented in bridge seismic design in Japan, that is established on the experience of HANSIN earthquake. The comparison of the major parameters in the specifications of China and Japan is shown in **Table 10.**

Table 9. Design Period (years)

Earthquake level / Performance level	E1	E2
A	475	2000
B,C	50-100	475-2000
D	25	--

Table 10. Comparison of main parameters in the seismic specifications

Specification	China JTJ044-089	China JTG/T B02-01-2008		Japan JRA-2002
Performance level	4 classes by importance coefficient	A、B、C、D		A、B
Site classification	I、II、III、IV*	I、II、III、IV**		I、II、III**
Areal coefficient (reflects seismic intensity)	Magnitude 6:simply performance Magnitude 7：0.1 Magnitude 8：0.2 Magnitude 9：0.4	Magnitude 6：0.05g Magnitude 7：0.10g or 0.15g Magnitude 8：0.20g or 0.30g Magnitude 9：0.40g		A zone：C_Z=1.0 B zone：C_Z=0.85 C zone：C_Z=0.7
Importance modified coefficient	1.7 1.3 1.0 0.6	E1 1.0 0.43(0.50) 0.34 0.23	E2 1.7 1.3(1.7) 1.0 Non	Embodied in evaluation of seismic resistance of the two performance levels (A or B).
Damping modified coefficient (C_d)	No mentioned	$C_d = 1 + \frac{0.05 - \xi}{0.06 + 1.7\xi} \geq 0.55$ Commonly ξ =0.05， So C_d =1.0		$C_D = \frac{1.5}{40h + 1} + 0.5$ When h=0.05, C_D =1.0
Definitely analytic method	Single mode response spectrum method	Multimode response spectrum method		Seismic coefficient method Ductility design method Dynamic analysis

Note:* Definited by the description of the properties, the component and the allowable bearing capacity of the foundation soil layers;

** Definited by the average velocity of the shear wave transmitting in the soil layers and their thickness.

4. DESCRIPTION OF SEISMIC ANALYSIS PROCEDURE

T shaped continuous beam bridge has several significant advantages: various choices of span and flexible applicability (20m,25m,30m and 40m per span is widely used), saving cost, convenient structure for lighter erection weight, abundant experience and good travel comfort. Due to these advantages, it is in large amounts structured in high level highway in combination with the actual circumstances in Yunnan. In common case the proportion can have been over 60% in total bridges.

The K138+800 bridge lies in the contract B15 of Qiliqing section along Yuanmou~Wuding expressway that is a section of Lanzhou~Mohan highway as the backbone highway for the Develop Western Regions Strategy. It has been open to traffic on Nov. 28th 2008. As the **Fig.10.** shown, its superstructure is T shaped continuous beam and 7 spans (30m per-span) with a total length of 218m and width of 24.5m, and is two-way 4 lanes by two separated structures for each way. It lies in long radius plan and profile curve of the route. The superstructure in each way is continuous beam for travel comfort and with 2 FD-80 expansion joints is connected with the gravity abutments in both sides. It can be seen in **Fig.11.**, the substructure is twin circle columns and piles, whose diameter are 1.5m and 1.6m respectively. The two columns connected by the bent cap and the tie beam form framework in the cross bridge axial direction. The reinforcement detail for column and pile is shown in **Fig.12.**

The K138+800 bridge has broadly representative as far as its span, form of components, dimension of column and geology condition concerned. The left way is selected as the subject to analyze. Furthermore the structure in the cross bridge axial direction has stronger seismic resistance than it in the bridge axial direction, in this paper only the resistance in the bridge axial direction is evaluated and all columns will be looked as socle beams.

(1)Model building

The structure is discretized and the computation module is built as **Fig.13.**

The 5 beams are replaced to a linear beam. And as shown in the **Fig.14.**, the 10 rubber bearings under the end of each span on the bent cap on column are replaced to the 5 linear spring elements in horizontal direction to link the beam with the bent cap. The horizontal spring stiffness K (kN/m) is defined by the following equation:

$$K= G_d\, A_r/\sum t \qquad (3)$$

Where G_d is the dynamic shear modulus of the bearing (kN/m^2), A_r is the shear area (m^2), $\sum t$ is the total thickness of all rubber layers of every support (m). The 5 sliding plate bearing under the end of the superstructure on the cap beam of the abutments are replaced to 5 vertical general supports. The deformations of all rubber bearings on the same bent cap or the cap beam are considered equal since only the bridge axial direction horizontally is evaluated.

For linking the beam with the bearing, and the column with the bent cap, the rigid linking style is used. The height difference of the two columns under the same bent cap cased by deck transverse slope is ignored. Every node of the pile is elastically supported in two orthogonal horizontal directions whose spring stiffness is obtained by the characteristic of the earth layers around the pile according to the <Japan JRA-2002>, because there is still not related provisions in the <China JTG/T B02-01-2008>. Every bottom of the piles is generally supported vertically.

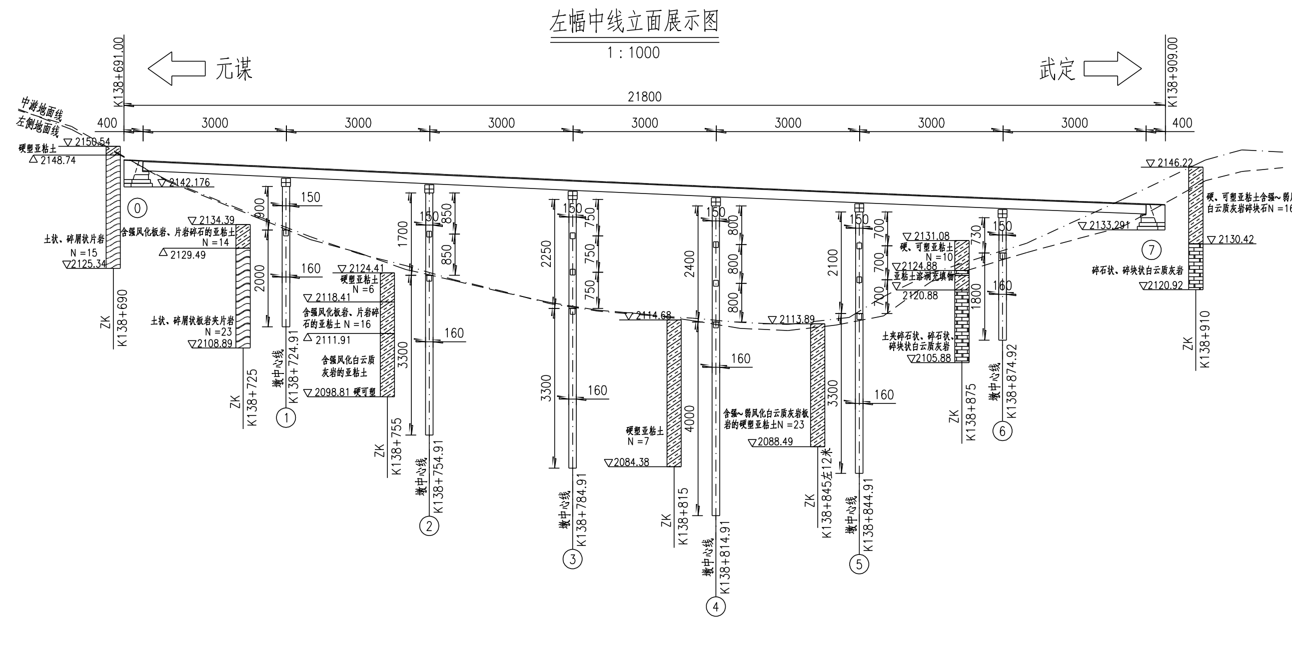

Fig.10. Outline of the K138+800 bridge

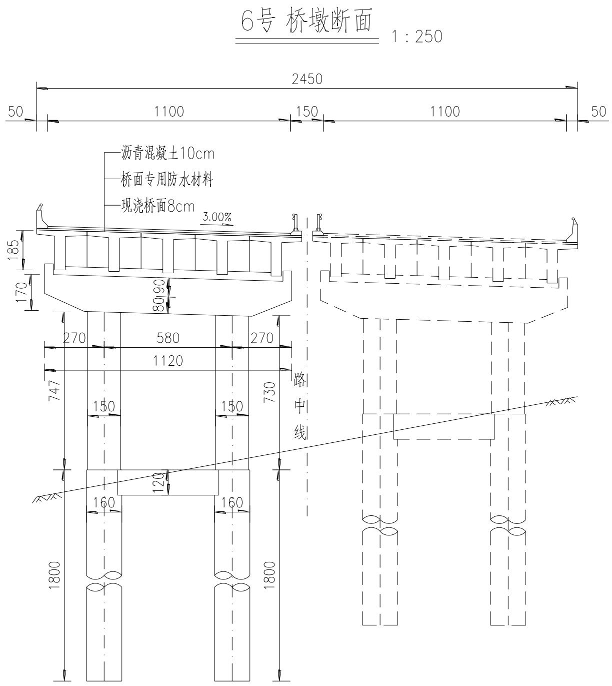

Fig.11. Cross section profile (The No.6 column)

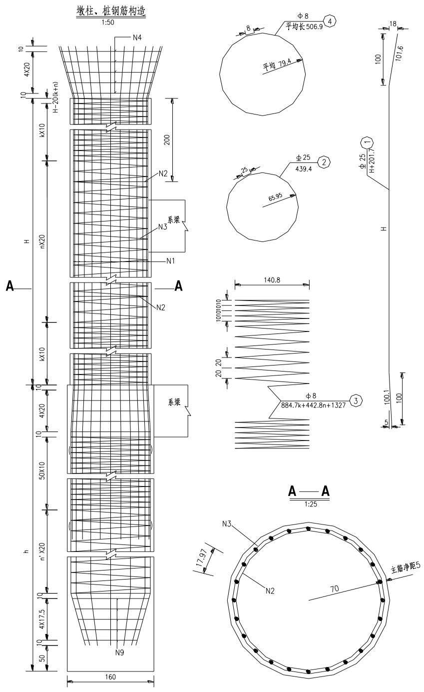

Fig.12. The reinforcement detail for column and pile

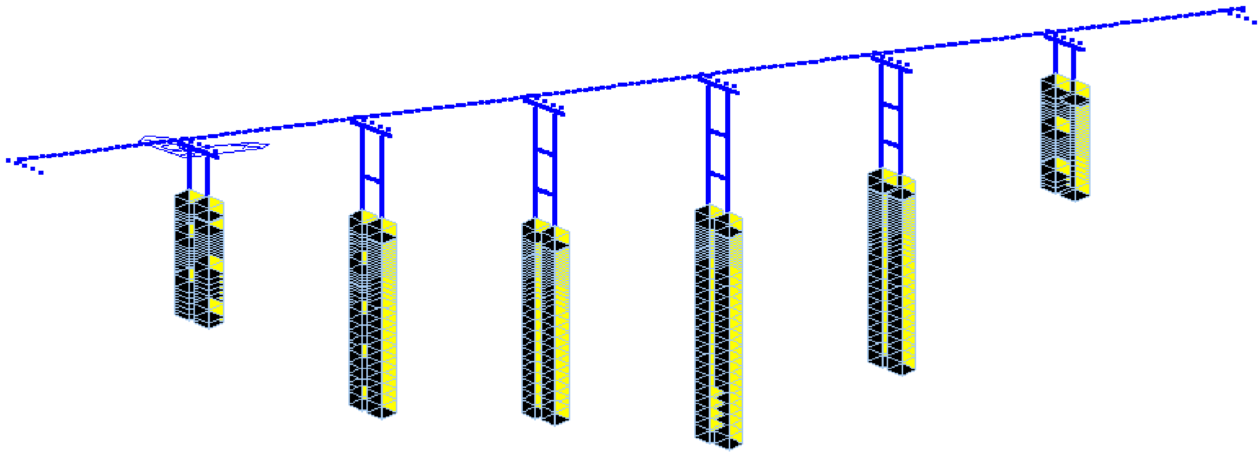

Fig.13. The discretized model by finite elements (elastic linking isn't shown)

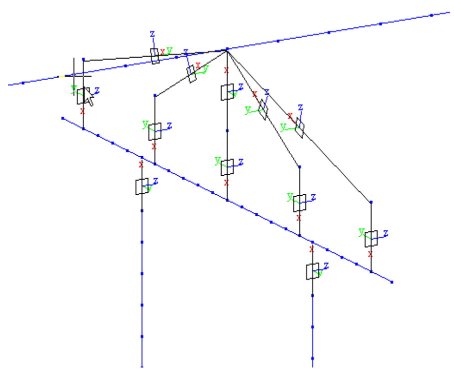

Fig.14. Linkage of between superstructure and Substructure

(2)Inelasticity Characteristic

Only the elastic stage of a structure can be calculated by the <China JTJ044-089>. As a revised version, the <China JTG/T B02-01-2008> still has no explicit relational expression between stress and strain for reinforcing steel bar and confined concrete when they are considered in nonlinear stage, so that in this paper this functional equation provided by the <Japan JRA-2002> is used. Moreover there is difference of the definition of concrete strength between Chinese specification that use 150mm×150mm×300mm cuboid as standard test piece and Japanese specification that use φ150×300mm and φ100×200mm cylinder as standard one. The designed standard strength of the C30 concrete of China is equivalently converted to 24(MPa) according to the Japanese Specification concerned.

Table 11. Characteristic values of C30 confined concrete and reinforcing steel bar

Specification	China JTG/T B02-01-2008	Japan JRA-2002	
		Type 1	Type 2
Axial compressive strength of the concrete (MPa)	20.1	24	
Ultimate strain of the confined concrete ε_{cu}	0.00558	0.0024633	0.0027274
Peak stress of the confined concrete f'_{cc} (MPa)	25.125	25.322	
Reduced ultimate strain of stirrup $\boldsymbol{\varepsilon}_{su}^{R}$	0.09	No mentioned	
Reduced ultimate strain of longitudinal tensile steel bar ε_{lu}	0.10	No mentioned	

Table 12. Characteristic value of the cross section of the bottom of the No.6 column as an example

Specification	China JTG/T B02-01-2008	Japan JRA-2002	
		Type 1	Type 2
The defined length of plastic hinge(m)	0.916	0.750	
Yield curvature of cross section ϕ_y (1/m)	0.0024712	0.0024830	0.0024830
Ultimate curvature of cross section ϕ_u (1/m)	0.0177500	0.0079692	0.0088551
Allowable maximum angle of rotation of the plastic hinge (rad）	0.0069996	0.0054910	0.0045726
Allowable maximum displacement of the top of the column (m)	0.1298	0.0915	

In the <JTG/T B02-01-2008>, some isolated design formulas such as the ultimate value of confined concrete and reinforcing steel bar, and the limit values of various nonlinear stages of the cross section of structural member bar are provided. As far as this example evaluated, the results by calculation are shown in **Table 11.** and **Table 12.** Obviously it is hard that the structural nonlinear analysis process is reliably implicated especially under the E2 level earthquake only by use of these results.

As shown in **table 12.**, the yield limit of confined concrete defined in the <China JTG/T B02-01-2008> and the <Japan JRA-2002> are similar, but the ultimate limit defined in the <Japan JRA-2002> is more safety. In terms of the allowable maximum of the angle of rotation of the plastic hinge and the displacement of the top of the column, the provisions in the <Japan JRA-2002> is also more safety.

(3)Major parameters involved

(a)The <China JTJ044-089>

The earthquake effect to the structure is calculated by the general influence coefficient in **Table 13.**

Table 13. General influence coefficient (C_Z)

Serial number of columns	1#	2#	3#	4#	5#	6#
Height of columns (m)	10.85	18.85	24.35	25.85	22.85	9.15
General influence coefficient	0.33	0.33	0.35	0.35	0.35	0.30
Note: The height of column is defined from the bottom of column to the surface of the padstone on bent cap.						

(b)The <China JTG/T B02-01-2008>

The acceleration response spectrums of the design earthquake E1 and E2 are shown in **Fig.15.** and **Fig.16.** and their maximum values respectively are E1: S_{max} = 0.1688 and E2: S_{max} = 0.5737. The first 50 vibration mode shapes are combinated by the CQC method.

(c)The <Japan JRA-2002>

The bridge is considered belonging to B category. The modification factor for zones is 0.7 for it also is artificially considered the example lies in C region and the foundation ground belongs to III category for the characteristic value of ground T_G = 0.627(s).

The natural period under the level1: T=2.375(s), the related design horizontal seismic coefficient K_h = 0.15。

The natural period under the level2: T = 3.221(s), the related design horizontal seismic coefficient: K_h = 0.51 (Type I) and K_h = 0.42 (Type II)。

The data from the special seismic investigation and safety evaluation for the bridge site is shortage, so the standard earthquake wave record T2-III-1 recommended by the <Japan JRA-2002> is artificially used for dynamic analysis (Time-history method) after it is modified by the modification factors (C_z) for zones of 0.7.

(4)Structural analysis by low scale earthquake

The structure is considered to keep in elastic stage under the low scale earthquake and the calculation result is shown in **Table 15**. Moreover the rubber support deformation is unnecessary to carry out the safety evaluation under the E1 effect according to the <China JTG/TB02-01-2008> and it is done only under the E_{hp} effect according to the <China JTJ044-89> as shown in **Table 14.**

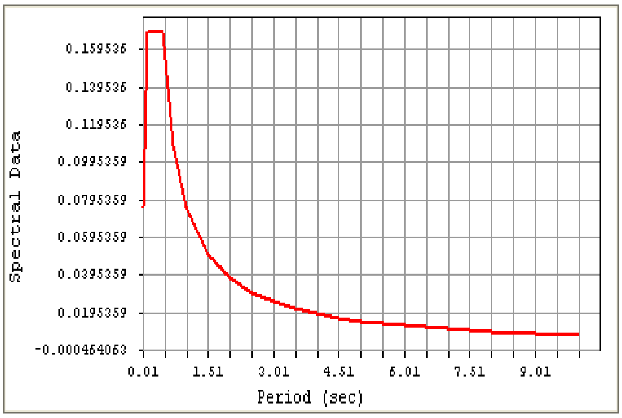

Fig. 15. Response spectrums of E1

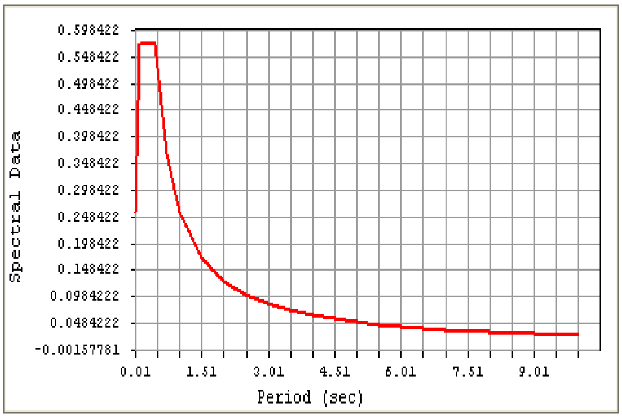

Fig.16. Response spectrums of E2

Table 14. The rubber bearing deformation (cm)

Bearing location (the S.N. of column related)	Deformation	Bearing location (the S.N. of column related)	Deformation	Safety evaluation
1	2.8	4	0.3	The permissible maximum deformation of this kind of the rubber bearing is 7.7cm, so they're OK!
2	0.9	5	0.5	
3	0.4	6	2.9	

Table 15. Calculation result under low scale earthquake effect

S.N. of column	Axial pressure (caused by gravity)	E_{hp} <China JTJ044-89>		E1 <China JTG/ TB02-01-2008>		Resistance of the related cross section related with E_{hp} or E1*		
		Moment of the bottom cross section	Shearing force of the bottom cross section	Moment of the bottom cross section	Shearing force of the bottom cross section	Moment resistance of right section	Axial Resistance of right section	Shearing resistance of inclined section
	(kN)	（kN·m）	(kN)	（kN·m）	(kN)	（kN·m）	(kN)	(kN)
1	4497.7	2794.2	281.8	2676.6	270.7	4682.9	6944.9	1883.0
2	4390.8	1993.9	114.3	1916.8	118.5	4867.9	7907.0	1875.5
3	4870.8	1553.3	71.5	1411.5	81.5	5089.0	9548.8	1909.1
4	4918.7	1422.3	62.8	1301.2	77.2	5131.9	10038.8	1912.5
5	4658.7	1699.0	82.3	1549.0	90.6	4986.9	8680.4	1894.3
6	4426.9	2400.0	291.5	2536.7	309.2	4972.5	8576.5	1878.1

Note：*The resistance force of section is calculated by the Chinese <Code for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts> JTG D62-2004。

（continue）

Level1 (Japan JRA-2002)		Resistance of the related cross section related with level1		
Moment of the bottom cross section	Shearing force of the bottom cross section	Moment resistance of right section	Axial Resistance of right section	Shearing resistance of inclined section
（kN·m）	(kN)	(kN·m）	(kN)	(kN)
13007.6	1338.6	2523.8	857.5	1883.0
9446.9	592.0	2670.3	1155.5	1875.5
7069.6	392.9	2929.7	1707.0	1909.1
6565.5	359.6	2986.1	1831.5	1912.5
7648.7	434.8	2840.3	1513.3	1894.3
12555.2	1548.0	2536.7	883.5	1878.1

It can be concluded from **Table 15.** as follows：

(a)The earthquake force provided by the <China JTJ044-89> is approximately equal to that by the E1 in the <China JTG/TB02-01-2008>. In other words, the earthquake effect provided by the China old seismic specification just equals to the low scale earthquake provided by the revised one；

(b)All columns are safety under the E_{hp} provided by the <China JTJ044-89> and the E1 by the <China JTG/T B02-01-2008>, the safety factors of the resistance of moment, shearing force and axial force are more than 1.5,1.5 and 6 respectively. Meanwhile the structure is beyond the limit of elastic stage of columns under the Level 1 provided by the<Japan JRA-2002> for the external moment and shearing force nearly are 2~5 times of the resistance of elastic stage, that is considered as not enough and unsafe seismic resistance.

(5) Structural analysis by strong earthquake by static method

The structure is considered to enter plastic stage under a strong earthquake and the calculation result is shown in **Table 17**. Moreover the rubber support deformation is safety evaluated under the E2 according to the <China JTG/TB02-01-2008> as shown in **Table 16.**

Table 16. Support deformation evaluation under E2 effect

Support location (the S.N. of column related)	Deformation (m)	Safety evaluation
1	4.9	The permissible maximum deformation of this type of rubber support is 7.7cm, so it's OK!
2	2.0	
3	0.7	
4	0.5	
5	0.9	
6	6.0	

Table 17. The evaluation of the No.6 column under strong earthquake

Specification	<China JTG/TB02-01-2008>	<Japan JRA-2002>	
Seismic effect	E2	level2	
Analysis method	Multimode response spectrum method	Ductility capacity method (push-over method)	
		Type1	Type2
Calculation result (take the No.6 column as example）	The shearing force at the bottom of column: V_{c0}=626.7(kN)<V_R=1590.2(kN); Maximal angle of hinge rotation: θ_p=0.0193(rad)> θ_u=0.0070(rad) Residual displacement of the top of column: δ_R=0.207(m)>δ_{Ra} =0.130(m)	Ductility capacity: $P_a = 529.5(kN) < k_{he}W = 2058.9(kN)$ Shearing resistance: $P_s = 502.4(kN)$ <Pa< $P_{so} = 704.7(kN)$ Residual displacement of the top of column: δ_R=0.241(m)>δ_{Ra} =0.092(m)	Ductility capacity: $P_a = 525.1(kN) < k_{he}W = 1713.4(kN)$ Shearing resistance: P_a <Ps=604.0(kN) Residual displacement of the top of column: δ_R=0.126(m)>δ_{Ra} =0.092(m)
Partial safety evaluation	Shearing resistance: OK！ Brittle failure can be avoided. Maximal angle of rotation: NO! Residual displacement of the top of column: NO！	Ductility capacity: NO！ Residual displacement of the top of column: NO！ Destructional forms: shearing failure after flexural yielding.	Ductility capacity: NO！ Residual displacement of the top of column: NO！ Destructional form: flexural failure
Note:(a)Every column is considered as cantilever beam in the bridge axial direction and the height of the No.6 column is the smallest, so it is the most strongly effected and the plastic hinge who the most early happened will come into being at the bottom of the No.6 column. So it is considered as the representative of all columns. (b)According to the <China JTG/T B02-01-2008>, the shearing force of the column is defined by selecting the smaller one between the result calculated by ability protect theory that has considered strengthen factor 1.2 and the result calculated by the E2 earthquake force. Meanwhile the shearing resistance is reduced by the safety factor 0.85. Both of them guarantee the principle of "strong shear capacity and weak bending capacity" and implement the Ability Protection for shearing resistance. The approaches are different, but the purposes are similar with the <Japan JRA-2002> that used different 2-level allowable ductility factor to "protect" the shearing resistance.			

(6) Structural analysis by strong earthquake by Time-History method

From the conclusion of **4.(5)**, the restriction at some level to the displacement of superstructure by abutments is considered to tap seismic potential of the structure and obtain more realistic result.

The restriction from the gravity abutments in both sides and the earth filled behind them are replaced at the abutment locations to two horizontal linear springs with an 8cm gap between the superstructure and the spring according to the size of expansion joints there. To every column three stages elastic-plastic mechanical strength characteristic is assigned. The potential plastic hinges also are assigned to the bottoms of every column.

The model is calculated by the T2-III-1(1995, HYOUGOKEN_South, N12W). When the assumptive largest displacement of the top of the No.6 column reaches 12.7cm (the given displacement superposed by the gaps from the expansion joint and the experiential deform of abutment), moreover the maximum moment of the plastic hinge at the bottom of the No.6 column is controlled: M_{max} =4522(kN·m) < M_u =4844.8(kN·m) by the adjustment of the spring stiffness of between the

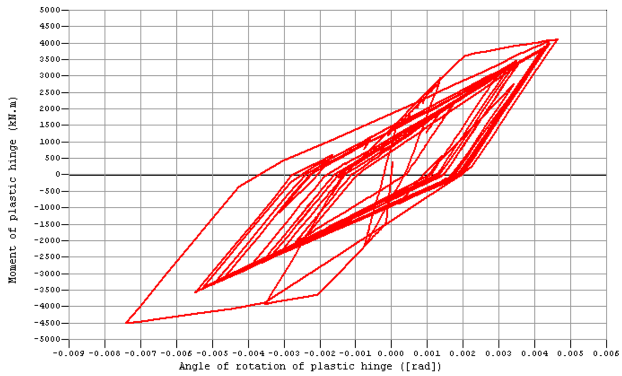

Fig.17. Skeleton curve between moment and angle of rotation of the plastic hinge at the bottom of the No.6 column

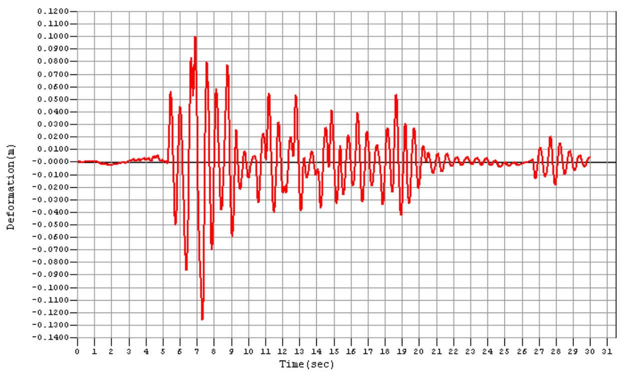

Fig.18. Time-history of the deformation at the rubber support on the No.4 column

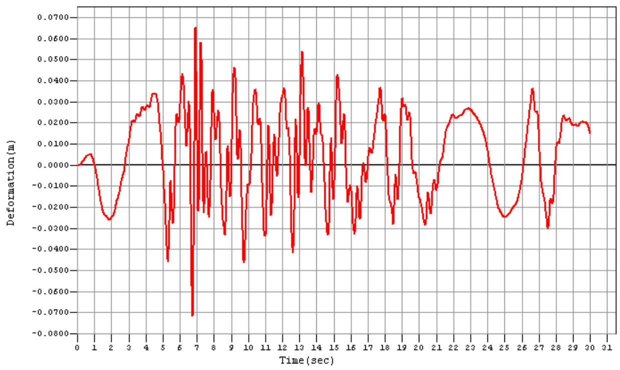

Fig.19. Time-history of the deformation at the rubber support on the No.6 column

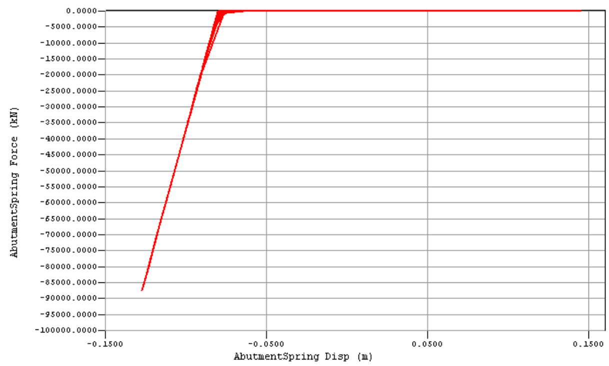

Fig.20. Time-history of the spring force between superstructure and abutment

superstructure and the abutments as shown in **Fig.17.**.

Meanwhile, the time-history skeleton of deformations of the rubber support that lie on the bent cap upon the No.4 and No.6 column are typically shown in **Fig.18.** and **Fig.19.** From them it can be seen that the maximum deformation on the No.4 whose column is the highest is 12.5cm and beyond the permissible maximum value of 7.7cm, while that on the No.6 whose column is the shortest is 7.1cm and within the permissible range. However the maximum force of the spring mentioned has reach 87420 (kN) as in **Fig.20.** shown. The huge acting force (impulse) has gone well beyond the capacity for acceptance of not only the abutments but also the ends of the superstructure. It is considered the abutments though can restrict the displacement of the superstructure and reduce the moment of column at some level, it cannot improve the seismic ability of the structure to reach safety level and the structure still is in dangerous under this kind of strong earthquake.

5. CONCLUSION

1)The seismic performance level of the bridges whose seismic design was implemented before Oct 1st 2008 according to the <China JT044-89> only equals to the performance for the low scale earthquake (E1) provided in the <JTG/T B02-01-2008> and cannot meet the requirement of the 3-level performance. It also has not provisions of the checking for displacement, the guarantee for ductility ability and the concept for the ability protection. The provisions in it concerning seismic detail design and resistant measure is unambiguous. For reason given above, the seismic resistance of many large highway bridges in Yunnan are not enough, including many expressway bridges accomplished not long ago and there is possibly potential safety hazard especially when a strong earthquake happens.

2) In strong earthquake conditions, the moment resistance of the column is less than the moment caused by earthquake, while the shearing resistance is enough. As such, the failure pattern is deduced as flexural failure and the brittle failure form can be avoided, that is considered is more reasonable for

safety. When the plastic hinge has come into being in the bottom of some columns, at least one of them the horizontal displacement of the top and the maximum angle of rotation of the hinge have gone beyond the maximum limit of both of the two specifications. It is also concluded that the large deformation will cause the bridge function failure.

3) The Chinese seismic structural theory and specification that is still imperfect in comparison with the Japanese one, it should widely borrow ideas from foreign advanced theory and experience and been continuously supplemented and improved. On the other hand, the bridges that have been constructed or completed to traffic should be in a planned way and with focuses seismic recalculated, checked and reinforced according to the new <JTG/T B02-01-2008> and some foreign reasonable successful experience, especially the bridges in the "lifeline" highway lie on higher seismic intensity zones.

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