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Seismic analysis of half-through steel truss arch bridge considering superstructure

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Abstract. This paper takes a half-through steel truss arch bridge as an example. A seismic analysis is conducted with nonlinear finite element method. Contrast models are established to discuss the effect of simplified method for main girder on the accuracy of the result. The influence of seismic wave direction and wave-passage on seismic behaviors are analysed as well as the superstructure and arch ring interaction which is mostly related with the supported bearings and wind resistant springs. In the end, the application of cable-sliding aseismic devices is discussed to put forward a layout principle. The main conclusions include: ① The seismic response isn't too distinctive with the simplified method of main girder. Generally speaking, the grillage method is recommended. ② Under seismic input from different directions, arch foot is usually the mostly dangerous section. ③ Vertical wave input and horizontal wave-passage greatly influence the seismic responses of arch ring, significantly increasing that of midspan. ④ The superstructure interaction has an obvious impact on the seismic performance. Half-through arch bridges with long spandrel columns fixed has a less response than those with short ones fixed. And a large stiffness of wind resistant spring makes the the seismic responses of arch ring larger. ⑤ A good isolation effectiveness for half-through arch bridge can be achieved by a reasonable arrangement of CSFABs.

Keywords: half-through steel truss arch bridge; seismic performance; cable-sliding friction aseismic bearing

1. Introduction

Half-through steel truss arch bridge is an important form of long-span arch bridge and this bridge type has a lot of inherent advantages. However, this type of arch bridge is much less common than deck-type arch bridges and tied-arch bridges, especially in the high-intensity areas. This is because of its seismic responses different from the general arch bridge. And the differences mostly result from the complex interaction between superstructure and arch ring. The flexible arch ring and rigid joints between the deck system and spandrel columns are the two major reasons

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caused the significant interaction. It makes the seismic analysis theory of half-through steel truss arch bridge far below mature level and limits its application in high-intensity areas. So this paper conducts a research on the seismic behaviors of half-through steel truss arch bridge with consideration of superstructure interaction as well as an effective seismic isolation design.

2. General situation of the bridge

Fig. 1 shows a plan view of the bridge to be investigated in this paper. It is a half-through steel truss arch bridge with a span of 519 m and a ratio of rise to span of 1/4. The arch ring has a catenary profile with a arch-axis coefficient of 4. The arch ring is composed by two parallel trusses with a spacing of 25.3 m. The spandrel columns are in the form of steel bent structure and the deck system is in the form of steel-concrete girder. There are bearings between the spandrel column 1#~3# and the girder, one side with fixed bearings and the other with horizontal sliding bearings; for the 4#~8#, one side with longitudinal sliding bearings and the other with bi-directional sliding bearings. Besides, there are four elastic springs devices (K=1000 KN/m) set between each joint of arch ring and deck system to aviod large longitudinal displacement of main girder under high wind speed. Three ground motion time histories synthesized from the local response spectrum with surpassing probabilities of 2% for 50 years are taken as seismic input. The peak acceleration for seismic input reaches 1.44 g. The displacement time histories can be obtained by direct integral of acceleration records which has been polynomial fitted to avoid the baseline drift from happening. Fig. 2 shows the ground motion input, including the acceleration time history, the placement time history and the acceleration response spectrum with surpassing probabilities of 2% for 50 years.

3. Analysis of simplified method for main girder

To study the correct simplified method of steel-concrete girder of the arch bridge in the finite element model, three FEM spacemodels with three different simplified methods of main girder are established. The deck system in Model One is modelled by one longitudinal girder and that of Model Two is modelled by grillage analogy, which are composed by vertical beam and cross beam, while longitudinal rigidity of bridge girder is taken by vertical beam and the lateral rigidity by cross beam. And Model Three uses plate element and beam element to simulate bridge decks

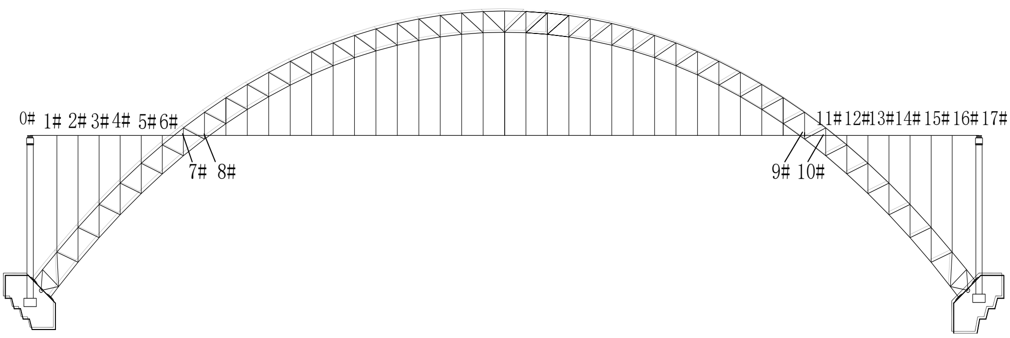
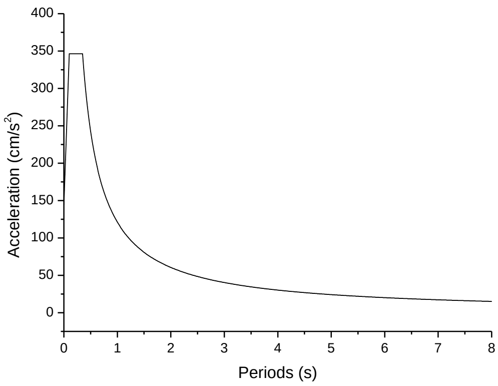
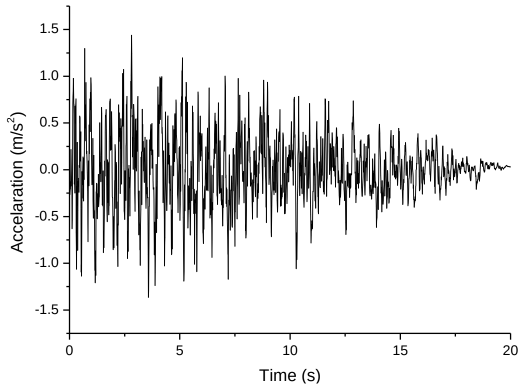


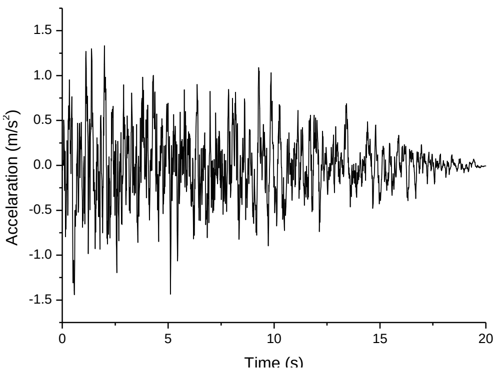
Fig. 1 The plan view of the bridge



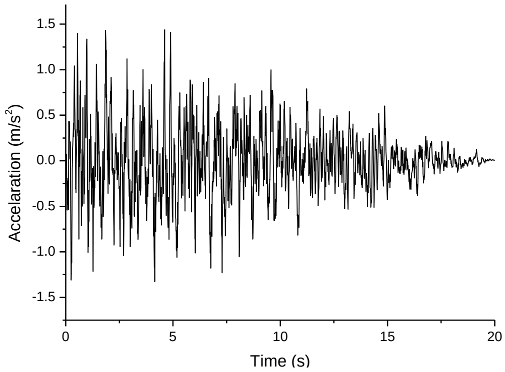
(a) The acceleration response spectrum



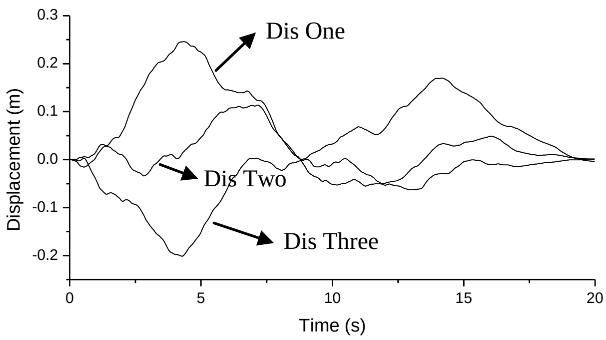
(b) The acceleration time history One



(c) The acceleration time history Two



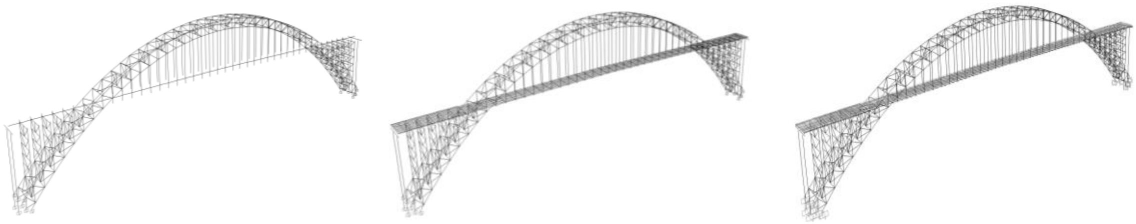
(d) The acceleration time history Three



(e) The displacement time histories with baseline adjusted

Fig. 2 The ground motion input

and steel girders, meantime sets rigid joints between the two parts to make them work together. The support reaction and suspender tensile in three models under dead load case are kept close to ensure that these models have girders of the same mass and transmitting force way. Fig. 3 shows the above three models and Fig. 4 shows the contrast of support reaction and suspender tensile in the three models. After the three models are established, mode analysis and time history analysis are applied to find the differences of the dynamic characteristics among the above three models.



Model One: single girder model Model Two: grillage analogy model Model Three: plate deck model

Fig. 3 The three FEM models with three different simplified method for main girder

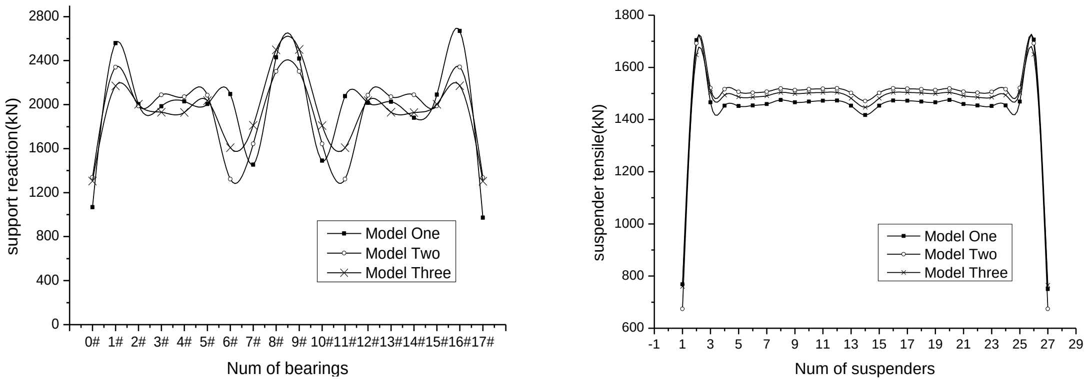
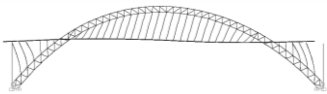


Fig. 4 The contrast of support reaction and suspender tensile in the three models

Table 1 Modes and periods which influence mostly

mode			
Describtion	axial drift of main girder	lateral curvature of arch and girder	vertical flexure of arch and girder
Model One	5.97s	4.14s	0.86s
Model Two	6.38s	4.51s	0.95s
Model Three	6.33s	4.28s	0.82s

3.1 Mode analysis of the three different models

The mode participating mass ratios shows that for Model One, modes which mostly influence the longitudinal, lateral and vertical seismic responses are the 1st, 2nd and 13nd mode respectively. And for Model Two, they are the 1st, 2nd and 14th mode. For Model Three, they are the 1st, 2nd and 9th mode. These modes and periods are listed in Table One. Results show that there are no significant differences of the modes which influence mostly among the three mode but the value of periods has a little differences.

3.2 Time history analysis of the three different models

When time history analysis is conducting, the following three cases are established. The three

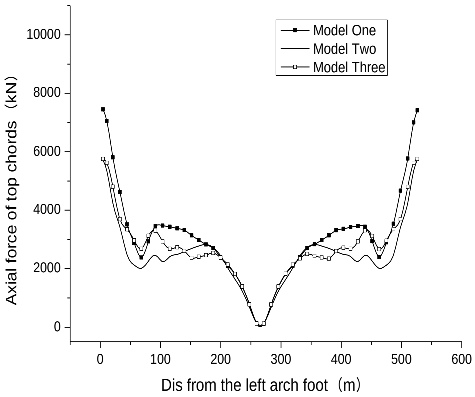


Fig. 5 Longitudinal wave input

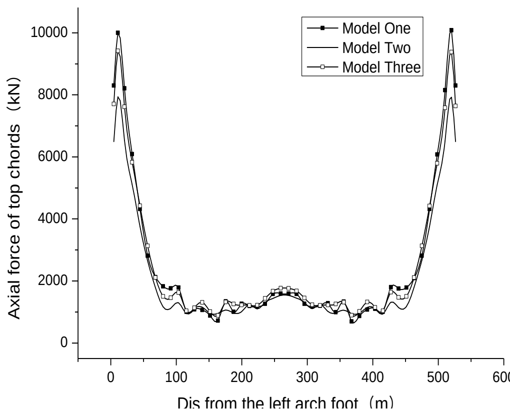


Fig. 6 Lateral wave input

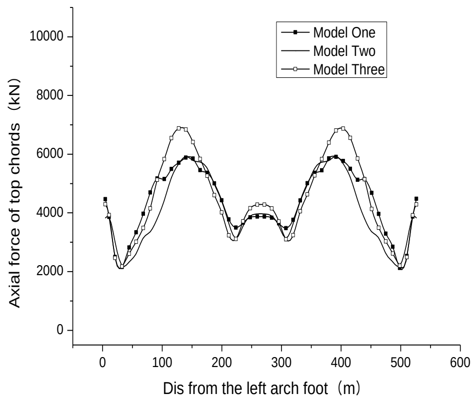


Fig. 7 Vertical wave input

acceleration time histories shown in Fig. 2 is performed and responses for the bridge under seismic ground motion is analyzed by Mode Superposition Method. A peak envelope is taken as the final result. Fig. 5, Fig. 6 and Fig. 7 shows maximum axial forces of top chords under seismic inputs from different directions.

Case1 longitudinal wave input,without considering traveling-wave effect

Case2 lateral wave input,without considering traveling-wave effect

Case3 vertical wave input,without considering traveling-wave effect

Results show that the main girder simplified method does have an effect on seismic behaviors. Under longitudinal wave input and lateral wave input, single girder model tends to response more strongly than others. And under vertical wave input, plate deck model tends to response more strongly.

4. Seismic analysis of half-through arch bridge

Three issues are discussed in this part to make a clear understanding about the seismic behaviors of half-through arch bridges. All the following results is based on the grillage analogy model.

4.1 Contrast of seismic responses under waves from different directions

In this part, two more cases are established besides for the above three cases. In the two cases, the peak acceleration for vertical wave input should be 2/3 of that for horizonal wave input. Fig. 8, Fig. 9 and Fig. 10 shows contrasts for maximum axial force of top chords under the five cases. Fig. 8 and Fig. 11 for the case 1, 2 and 3. Fig. 9 and Fig. 12 for the case 1, 3 and 4. Fig. 10 and Fig. 13 for the case 2, 3 and 5. All the load cases are analyzed by Mode-superposition Method.

Case4 longitudinal and vertical wave input,without considering wave-passage effect

Case5 lateral and vertical wave input,without considering wave-passage effect

It can be seen from figure 8 that under longitudinal wave input and lateral wave input, the axial force on arch foot is larger than that on mid-span or quarter-span and under vertical wave input, the axial force on quarter-span is larger than that on mid-span and arch foot. As for the peak axial forces for the whole ring, responses under lateral wave input is about 140% of that for the other two cases. But as for midspan axial force, responses under vertical input is about 30 times of that for longitudinal input and 2.5 times of that for lateral input. Besides, it can be indicated from Fig. 9 and Fig. 10 that vertical seismic input functioning with horizontal seismic input significantly increases midspan axial forces in comparation with single-dimension wave input, which is similar with long span deck-arch bridges.

Table 2 shows the shear forces of supported bearings under different seismic inputs. It can be seen that shear forces under lateral input are much larger than that under longitudinal input, even ecceeding the ultimate shear-resistant capacity of bearings. Besides, shear forces of bearings under lateral input are quite heterogeneous.

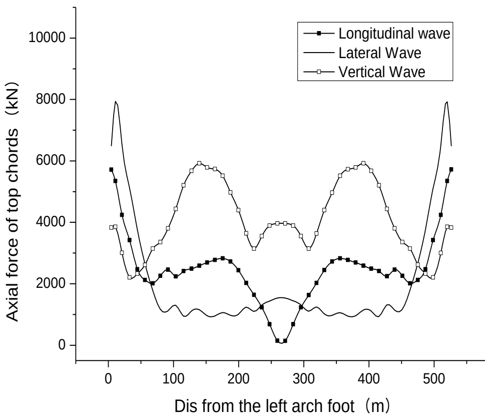


Fig. 8 Contrast One

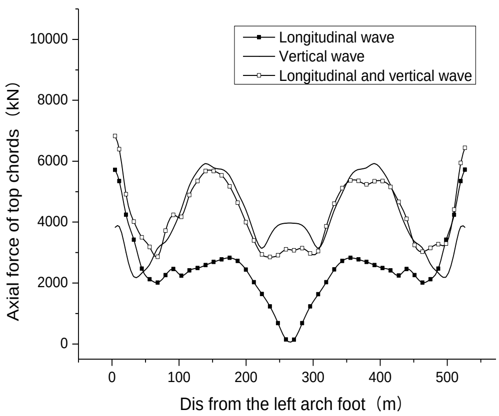


Fig. 9 Contrast Two

Table 2 The shear forces of bearings under different seismic input

Load Cases / Bearing Num		0#	1#	2#	3#	4#	5#	6#	7#	8#
Shear Force /kN	Longitudinal	60.8	47.3	57.2	82.1	60.4	60.6	60.6	60.6	80.8
	Lateral	642.5	1146.7	637.4	346.6	295.8	292.8	372.4	421.8	2131.0

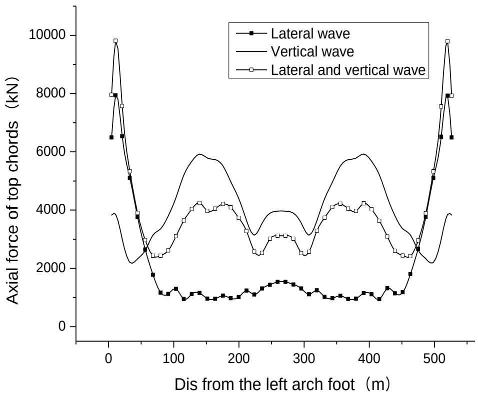
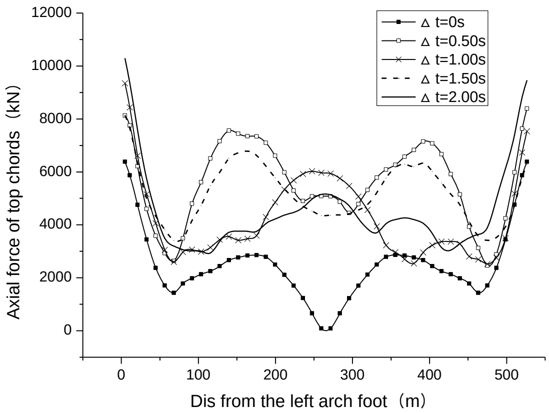


Fig. 10 Contrast Three



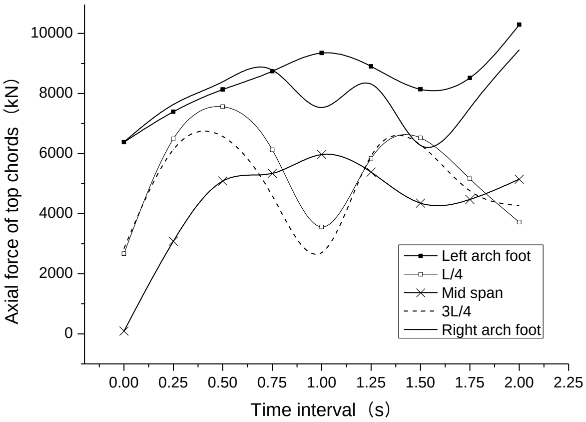


Fig. 11 Results under longitudinal seismic wave input considering wave-passage effect

4.2 Influence of wave-passage effect on seismic responses

As the half-through steel truss arch bridge is a structure system of high stiffness, wave-passage effect has a strong influence on its seismic responses, which is mostly bacause that several antisymmetric modes will be excited with an interval between the arriving time at left and right arch foot and only symmetric modes can be excited under uniform seismic ground motion.

To study the influence of wave-passage with different excitation directions and time intervals, models are established with the arriving time lag at left arch foot and right arch foot ranging from 0s to 2s in case that seismic wave travels from left to right. Fig. 11 shows the maximum axial forces of top chords under longitudinal excitation with different time lags as well as the increasing extent at different positions. Fig. 12 for the lateral excitation. In this part, all the time-history analysis is done by Direct Integral Method. Considering that the anti-shear capacity of bearings under lateral seismic ground motion is also difficult to satisfy requirements, Table 3 shows changes of shear forces.

Generaly speaking, wave-passage effect greatly influences seismic responses of arch ring. But the impact depends on positions, time intervals and wave directions. It can be informed from Fig. 11 that under longitudinal seismic input, responses at arch foots increases with time lag increase

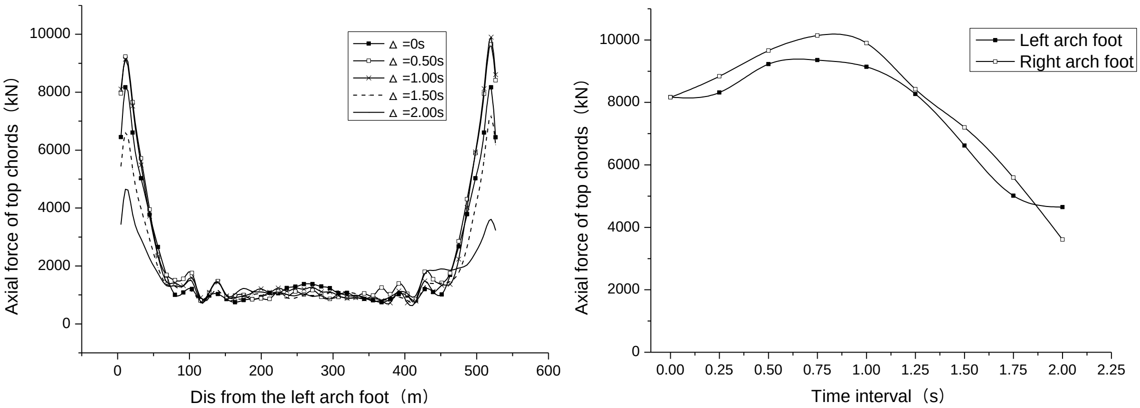


Fig. 12 Results under lateral seismic wave input considering wave-passage effect

Table 3 Shear forces of bearings under lateral seismic input (kN)

	0#	1#	2#	3#	4#	5#	6#	7#	8#
Δ *t*=0s	643	1147	637	347	296	293	372	422	2131
Δ *t*=0.25s	648	1153	640	348	291	288	372	429	2127
Δ *t*=0.50s	687	951	549	313	254	245	373	616	2631
Δ *t*=0.75s	808	844	422	281	254	270	431	798	2682
Δ *t*=1.00s	713	844	455	291	266	261	416	899	2256
Δ *t*=1.25s	699	944	507	274	253	240	417	813	2117
Δ *t*=1.50s	723	1097	557	281	241	223	381	708	1957
Δ *t*=1.75s	808	844	422	281	254	270	431	798	2682
Δ *t*=2.00s	770	927	462	210	201	196	258	549	1475

and a maximum increment reaches 160%. Besides, the responses at midspan also increase sharply, nearly 70 times in case of a time lag of 1.0s. As for lateral seismic input, it is the responses at arch foots that have an obvious change under wave-passage influence, an increase with time lags less than 1s and a decrease with time lags more than 1s, maximum increment reaching 130%. In addition, shear forces of bearings except 7# has less relationship to wave-passage effect, whose increasing amplitude is no more than 30%. But the shear forces of bearing 7# has a maximum increase of 100%, which should be paid attention at design time.

4.3 Influence of superstructure interaction on longitudinal seismic performance

Bearing arrangement and spring stiffness Ssp are two factors that effect the, so the following models are set up.Model I, Model II, Model III, Model IV and Model V are set up to compare the impact of bearings arrangement on seismic responses. Model I, Model VI, Model VII, Model VIII, Model IX and Model X are set up to investigate the impact of spring stiffness on seismic response. Longitudinal mode analysis and time history analysis under load case Two are conducted.

Model I 1#, 2# and 3# are longitudinal fixed ; S_{sp} is 1000kN/m

Model II	2#, 3# and 4# are longitudinal fixed ; S_{sp} is 1000kN/m
Model III	3#, 4# and 5# are longitudinal fixed ; S_{sp} is 1000kN/m
Model IV	4#, 5# and 6# are longitudinal fixed ; S_{sp} is 1000kN/m
Model V	All except 0# are longitudinal fixed; S_{sp} is 1000kN/m
Model VI	1#, 2# and 3# are longitudinal fixed ; S_{sp} is 2000kN/m
Model VII	1#, 2# and 3# are longitudinal fixed ; S_{sp} is 4000kN/m
Model VIII	1#, 2# and 3# are longitudinal fixed ; S_{sp} is 6000kN/m
Model IX	1#, 2# and 3# are longitudinal fixed ; S_{sp} is 8000kN/m
Model X	1#, 2# and 3# are longitudinal fixed ; S_{sp} is 0kN/m

Table 4 Seismic responses of models with different arrangements of bearings

Load Cases	the 1st longitudinal mode period /s	Axial force /kN			M_{max} for arch foot /kN·m	D_{max} for main girder /cm	D_{max} for bearings /cm	M_{max} for spandrel columns /kN·m
		0	L/3	L/2				
Model I	6.35	5701	2826	141	2005	11.7	13.5	2403
Model II	5.64	5769	2836	141	1991	12.2	13.1	3487
Model III	4.15	7190	2817	142	2348	10.5	7.0	7222
Model IV	3.19	11993	6007	172	4114	7.9	6.2	7586
Model V	3.19	12003	6011	172	4109	7.9	6.2	7590

4.3.1 Impact of bearing arrangement

Table 4 and Fig. 13 shows the changes of the first longitudinal modes and maximum structure responses under longitudinal seismic input with different arrangement of bearings. It can be indicated that models with long spandrel columns fixed have longer periods, larger displcements of main girder, weaker responses of arch ring and stronger responses of spandrel columns in comparation with those with short spandrel columns fixed. Besides, the differences are more distinct at arch foots or position that is L/3 from arch foots. Increasing

It can be analyzed from a perspective of energy. In the earthquake, one part of the energy will shift to the kinetic energy, and another is transformed into structure deformation energy, which is more dangerous to the structural seismic performance. Comparing with long columns, short columns have a higher stiffness and a stronger displacement-constrainting capacity. So fixing short columns and loosing long ones lead to a smaller displacement of main girder, meaning less kinetic energy, more structural seismic performance and larger seismic responses of arch ring. Considering that arch foots are usually controlled sections at design times, models with long spandrel columns fixed are suggested to be used in case that these models can all satisfy normal working requirments.

4.3.2 Impact of spring stiffness

The springs on the joints of deck system and arch ring are set to take force and limit displacement of main girder under longitudinal wind. But they also have an effect on the longitudinal dynamic characteristic and seismic responses. Table 5 and Fig. 14 shows the response results for models with springs of different stiffness. It can be seen that t spring devices with larger stiffness lead to a stronger constraint for the main girder, a shorter period for the whole bridge and

Table 5 Seismic responses of models with different spring stiffness

Spring Stiffness /kN·m^{-1}	the 1st longitudinal mode period /s	Axial force /kN			M_{max} for arch foot /kN·m	D_{max} for main girder /cm	D_{max} for bearings /cm	M_{max} for spandrel columns /kN·m
		0	*L*/3	*L*/2				
0	8.60	5575	2844	140	1967	9.4	13.5	2081(3nd)
1000	6.35	5701	2826	141	2005	11.7	13.5	2403(3nd)
2000	5.38	6029	2819	143	2069	11.5	11.1	2313(3nd)
4000	4.47	6632	2869	148	2209	10.2	7.5	1835(3nd)
6000	4.05	7381	3185	149	2524	9.7	6.5	1709(4th)
8000	3.82	7798	3471	152	2687	9.4	5.8	1743(4th)

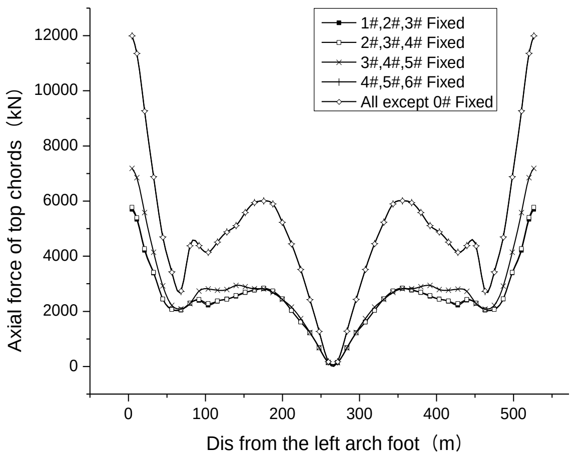


Fig. 13 Results of models with different layout of bearings

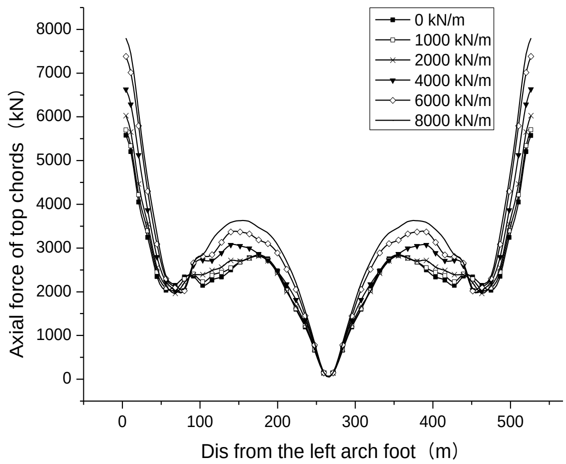


Fig. 14 Results of models with different spring stiffness

a shorter displacement for the deck system, meaning stronger seismic responses. So the spring stiffness should be carefully designed.

5. Seismic isolation design considering lateral seismic input

To find a reasonable seismic isolation design aimed at the lateral seismic responses of this kind of bridge, a new seismic isolation devices developed by Yuan Wan Cheng Professor research team, namely the cable-sliding friction aseismic bearing (CSFABs) , is applied in the seismic isolation design.

5.1 Introduction of cable-sliding friction aseismic bearing (CSFABs)

Cable-sliding friction aseismic bearings is a kind of cable isolation device and can turn to sliding bearings under strong seism so as to protect the main structure from damage. CSFABs are composed of general pot bearings and elastic cable. The restoring force model of bi-directional

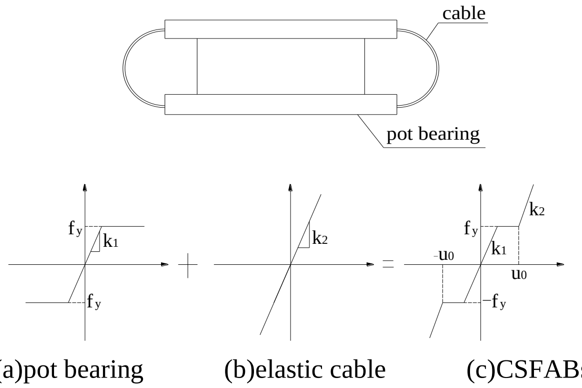


Fig. 15 The sketch and restoring force model of CSFABs

sliding pot bearing is similar to that of ideal elastic plastic materials and the restoring force model of elastic cable is a linear model. So the restoring force model of CSFABs can be considered as a combination of that of bi-direction sliding pot bearing and elastic cable. As shown in Fig. 15, k1 for the elastic stiffness of sliding pot bearings, fy for the critical friction force, u0 for the freedom placement and u for the sliding placement. The CSFABs work like this:

① when $u \le u0$, it works like general pot bearings;

② when $u \ge u0$,shear keys are snipped and cables begin to work.

This kind of seismic isolation device is suitable for bridges with large shear forces of bearings and great internal forces of main structure under seismic ground motion. It can reduce seismic responses by releasing the constraints between superstructure and infrastructure, as well as limiting the displacement of deck system to an allowed range. From a perspective of energy, more kinetic energy has be obtained so that less deformation energy is generated. Besides, CSFABs has a clear restoring force model and can be accurately calculated. Under lateral wave input, the bridge investigated has large internal forces of arch ring and large shear forces of bearings, which is excepted to be snipped with the usual design. So it is suitable to apply CSFABs to achieve a satisfied seismic performance. Or the strong seism will lead to too much internal force or too large displacement, causing enormous economic losses and casualty.

5.2 Introduction of models and cases to be analysed

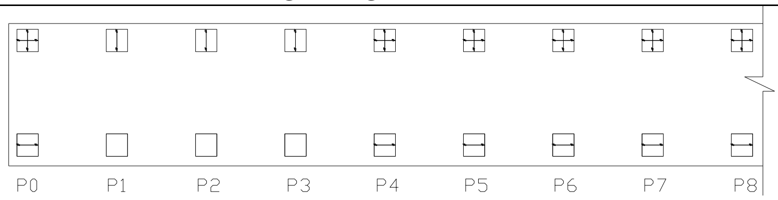
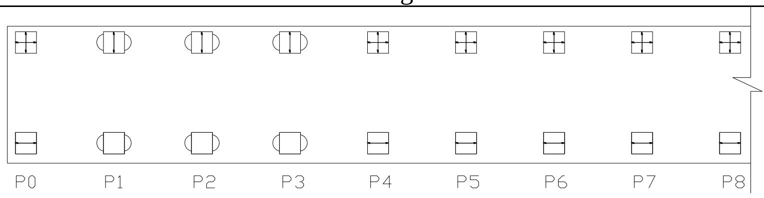
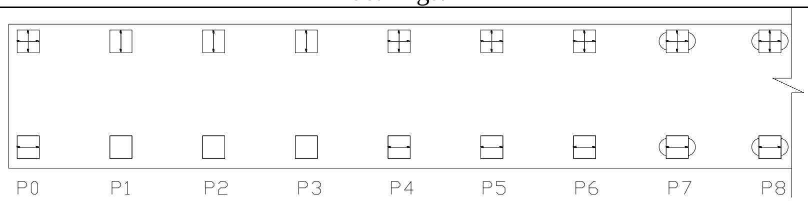
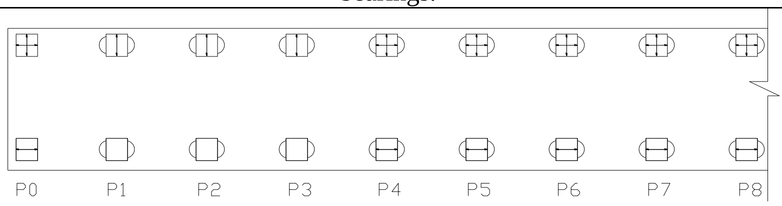
Four contrast models with different arrangement of CSFABs are established based on the grillage analogy model to study the reasonable layout of cable-silding seismic isolation devices for half-through steel truss arch bridge. Table 6 shows the four contrast models.

After four models are established, time history analysis are applied to study the differences of the seismic responses among these models. In this process, the following load case 6 and case 5 mentioned in section 2 are performed. The seismic fortification intensity at the bridge site is 8 degreen,and the peak acceleration of ground motion reaches 0.20 g according to the norm and the design request. So the peak acceleration for horizontal seismic input can be adjusted to 0.20 g while keeping spectrum characteristic the same.

Case 5 lateral and vertical wave input,no wave-passage, a peak acceleration of 0.144 g

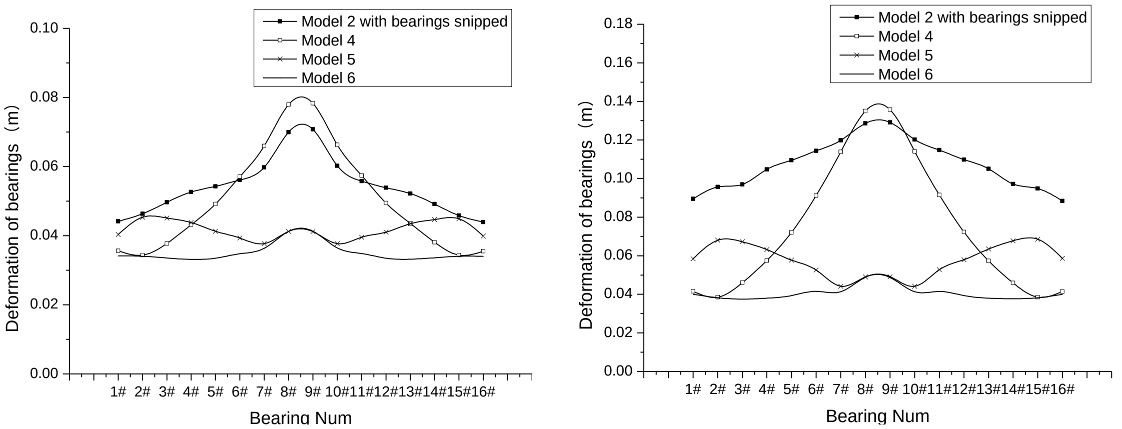
Case 6 lateral and vertical wave input, no wave-passage, a peak acceleration of 0.200 g.

Table 6 The contrast models with different arrangement of bearings

Num	Bearing Arrangement Sketches
Model 2	 P1~P3: one side with fixed bearings, the other with horizontal sliding bearings; P0, P4~P8:one side with longitudinal sliding bearings , the other with bi-directional sliding bearings.
Model 4	 P1~P3: one side with fixed CSFABs, the other with horizontal sliding CSFABs; P0, P4~P8:one side with longitudinal sliding bearings , the other with bi-directional sliding bearings.
Model 5	 P7~P8:Both the two side with CSFABs, P0~P6:one side with longitudinal sliding bearings , the other with bi-directional sliding bearings.
Model 6	 P1~P8:Both the two side with CSFABs, P0:one side with longitudinal sliding bearings , the other with bi-directional sliding bearings.

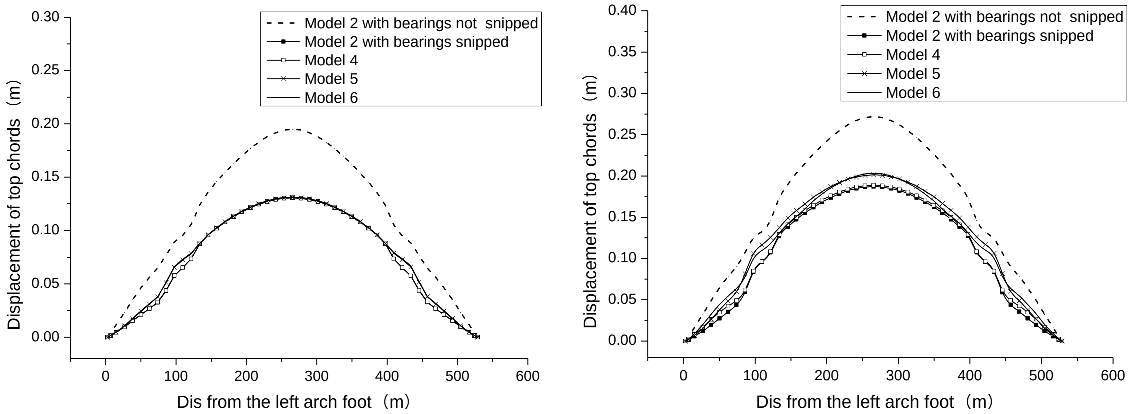
5.3 Analysis of seismic responses of the contrast models

Fig. 16 shows the bearing deformation of contrast models under lateral seismic input. Among these models, Model Two is on the promise that all the general fixed pot bearings of the models aren’t snipped to get maximum deformation. And the other models are on the cases that all the bearings are snipped and turn to bi-directional sliding bearings to ensure the cable works normally. Fig. 17 and Fig. 18 shows the displacement and axial forces of top chords on the arch bridge under earthquake motion input. Among these models, Model Two with bearings not snipped are established to get maximum internal forces in the progress and make a contrast with Model Two



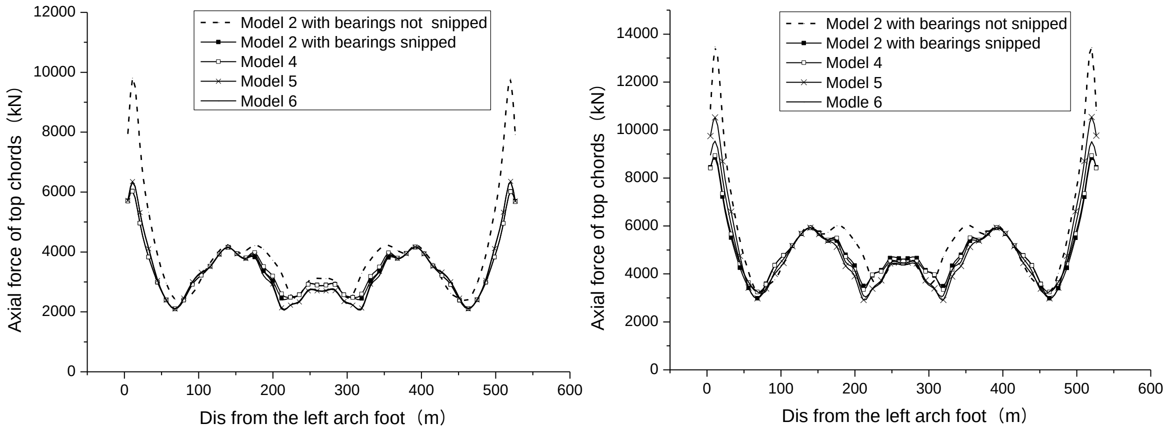
(a) Seismic wave with a peak of 0.144 g (b) Seismic wave with a peak of 0.200 g

Fig. 16 The deformation of bearings under lateral earthquake input



(a) Seismic wave with a peak of 0.144 g (b) Seismic wave with a peak of 0.200 g

Fig. 17 The displacement of top chords under lateral earthquake input



(a) Seismic wave with a peak of 0.144 g (b) Seismic wave with a peak of 0.200 g

Fig. 18 The axial force of top chords under lateral earthquake input with bearings snipped.

It can be seen from Fig. 16 that under seismic wave with a peak of 0.144 g, maximum bearing deformation of model 4 increases to 105% of that of model 2, maximum of model 5 reduces to 64% of that of model 3 and maximum of model 6 reduces to 58% of that of model 3. While under the wave input with a peak of 0.200g, the maximum for bearing deformation of model 5 reduces to 53% of that of model 3 and maximum for model 6 reduces to 38% of that for model 3. As to seismic responses, Fig. 17 and Fig. 18 shows that models with CSFABs has similar responses with the model with no CSFABs but all fixed bearings snipped, approximating 65% of that for models with fixed bearings not snipped.

The results shows that a reasonable application of CSFABs in half-through arch bridge can effectively limit the bearing deformation as well as release the constraints between superstructure and arch ring to protect the main structure from damage under lateral earthquake ground motion input, transforming more earthquake energy to kinetic energy. It's obviously that model 5 and model 6 work better than model 4 in aspect of displacement limitation and main structure protection. It indicates that the seismic isolation system with the CSFABs layed on the middle of deck system has a better effect than the seismic isolation system with the CSFABs layed on the end of deck system.

6. Conclusions

This paper takes a half-through steel truss arch bridge as an example. Space models are developed and dynamic analysis is performed. The responses of contrast models are compared to discuss the accuracy of different modelling methods and the influence of superstructure interaction on seismic performance, as well as the reasonable placement of cable seismic devices. The main conclusions from the analysis include.

- The simplified method for main girder has few influence on seismic responses as long as there is a correct transmitting force way between the superstructure and arch ring. Considering that the plate deck method costs too much time and it is difficult to get a correct transmitting force on the single girder method , grillage method is recommended, which can help engineers save time as well as guarantee calculation precision.
- Under seismic input from different directions, the positions where the strong responses happen are different but arch foot is usually a mostly dangerous section. Vertical wave input and horizontal wave-passage greatly influence the seismic responses of arch ring, significantly increasing that of midspan. It is suggested to focus on the arch foot as well as the impact of wave-passage when calculating the seismic repsponses of half-through arch bridges.
- The superstructure interaction has a significant impact on the seismic behaviors. Half-through arch bridges with long spandrel columns fixed has a stronger response than those with short ones fixed. And a large stiffness for wind resistant spring makes the the seismic responses of arch ring larger as well as reduce the displacement of the deck system. So it is suggested to make short spandrel columns fixed rather than long ones and choose reasonable stiffness for wind resistant considering seismic force and displacement together.
- By reasonably placement of CSFABs, a good seismic isolation effectiveness of the half-through steel arch bridge can be achieved. Comparing the seismic responses of different models, it is suggested the CSFABs to be layed near the middle of the main girder to get a better seismic reduction effectiveness.

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