Pre-Stressed Concrete Box Girder Bridge Hinge Vertical Restrainer Under Vertical Earthquake

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SUMMARY:
There are many cast in place pre-stressed concrete box girder bridges in California with inter span hinges. The length of the short and long part of these hinges were almost 8 to11 meters and 60 to 65 meters respectively. Conventional hinges consist of seat and upper seat that the long part of the hinge (long cantilever) will be supported by the short part of the hinge (short cantilever).

During the vertical component of an earthquake, long and short cantilever will be excited in different modes. In this paper besides considering the conventional pre-stressed concrete box girder bridges, a dynamic analysis on the vertical restrainers in the hinge section will be presented.

Keywords: Box girder bridge; pre-stressed cast in place concrete; vertical re-strainer, dynamic analysis.

1. INTRODUCTION

1.1 Types of the bridges

Most of the long (more than 5 spans) cast in place concrete pre-stressed box girder bridges in California have inter-span hinges almost after two consecutive spans. These hinges have short and long parts as shown in Figure 1.1.

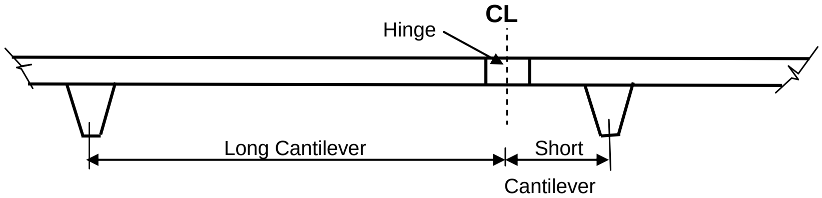

Figure 1.1. Long and short parts of a hinge

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Typical section of these kinds of hinges is illustrated in Figure 1.2.

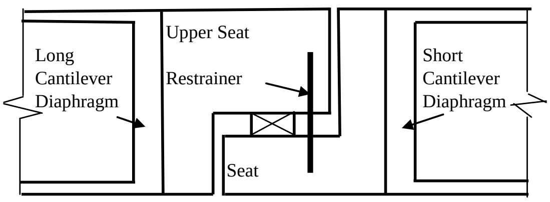


Figure 1.2. Long and short parts of a hinge

In order to minimize the vertical movement of these hinges, vertical restrainers will be often used by designers. During an earthquake, long and short cantilever will be excited in different modes under vertical component of earthquake. In this paper besides considering the conventional pre-stressed concrete box girder bridges, a dynamic analysis for vertical excitation on vertical restrainers in the hinge section will be presented.

2. **DYNAMIC ANALYSIS APPROACH**

There are six different ground accelerations during an earthquake, two horizontal, one vertical and three rotational as shown in Eqn. 2.1.

$$\{E\}T= \{Ügxi, Ügyi, Ügzi, Ü\theta gxi, Ü\theta gyi, Ü\theta gzi\} \tag{2.1}$$

These accelerations cause lateral and twisting forces on the masses, Eqn. 2.2.

$$\{F(t)\} = [M][r]\{E\} \tag{2.2}$$

[M] = Matrix of the mass,
[r] = Matrix of the earthquake influence.

A pre-stressed cast in place box girder bridge frame with two 65m and 74m spans has been considered for dynamic analysis. This bridge had five frames and four inter span hinges. The finite element model of this frame has 18 joints J#1 through J#18 and the first and last joints are hinges as it is shown in Figure 2.1. The J#18 (second hinge) has divided the 74m span to two cantilever beams. Short cantilever is 9m long and will be carried the 65m long cantilever reaction. Cross section area and moment of inertia of the deck were 10.93m2 and 14.19m4 respectively. Three different cases (1 through 3) will be considered to cover all the possible mass distributions. In case “1” no mass is considered at J#1 to cover the first frame of the bridge that will be supported with abutment at J#1. Fourteen masses were considered for whole length of two spans to represent the uniform mass of the bridge deck. Case “2” will represents the second, third and fourth frames that the left short cantilever will support the adjacent long cantilever and has small mass for the short cantilever before supporting the long cantilever at its left side. Case “3” will represents the second, third and fourth frames that the left short cantilever will support the adjacent long cantilever and has huge mass for the short cantilever supporting the long cantilever at its left side. Figures 2.2 and 2.3 are showing the cases “2” and “3” respectively.

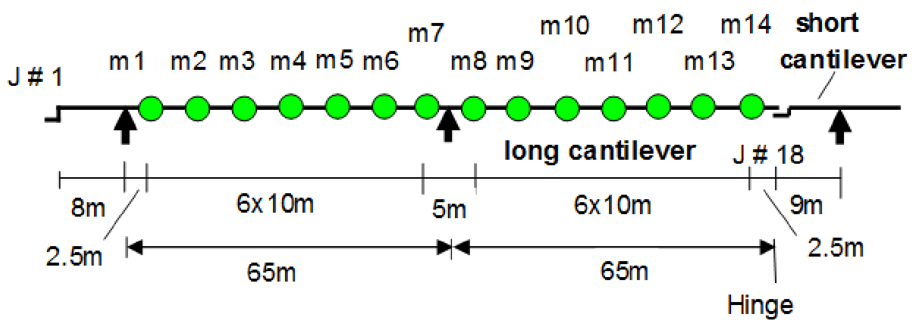


Figure 2.1. Bridge frame Case “1” with no mass at J#1

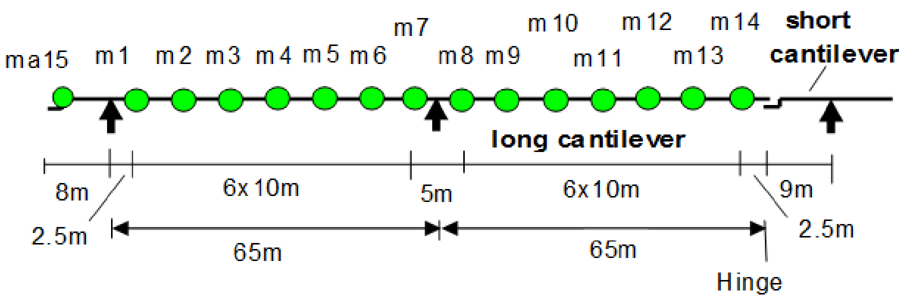


Figure 2.2. Bridge frame Case "2" with short cantilever mass at J#1

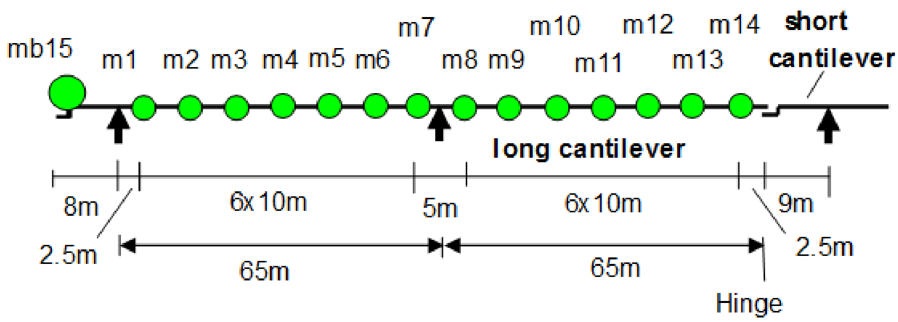


Figure 2.3. Bridge frame Case "3" with adjacent long cantilever mass reaction at J#1

The bridge frame deck joint masses "m1" through "m15" are shown in Table 2.1.

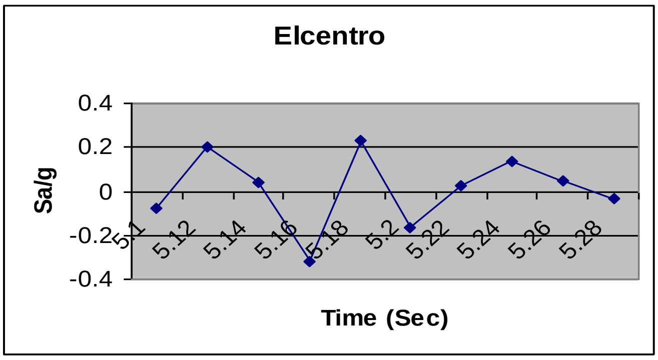
Table 2.1. Bridge deck joint masses

Mass	KN	Mass	KN
m1	16.4	m9	26.24
m2	26.24	m10	26.24
m3	26.24	m11	26.24
m4	26.24	m12	26.24
m5	26.24	m13	26.24
m6	26.24	m14	16.4
m7	16.4	ma15	26.24
m8	16.4	mb15	91.84

The El-Centro 1940 earthquake horizontal acceleration from time 5.1sec to 5.2sec that contains the highest accelerations are shown in Table 2.2. Vertical ground accelerations of an earthquake are mostly less than the horizontal ground accelerations. In this research the vertical ground acceleration is considered one tenth of the tabulated horizontal ground accelerations.

Table 2.2. Horizontal ground accelerations (El-Centro 1940)

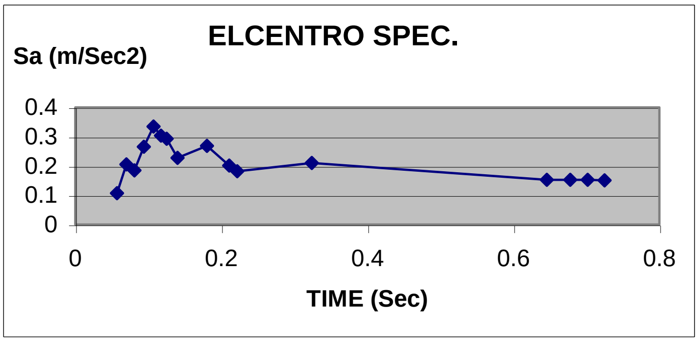
TIME(sec)	Sa/g
5.1	-0.0765
5.12	0.20135
5.14	0.04196
5.16	-0.3188
5.18	0.22981
5.2	-0.1666
5.22	0.02224
5.24	0.13759
5.26	0.04458
5.28	-0.0356



The El-Centro 1940 design response spectrum that is considered in this research is shown in Table 2.3.

Table 2.3. El-Centro 1940 design response spectrum

TIME	ω(rad/sec)	Sa(m/s^2)
0.0573	109.5	0.1072
0.0702	89.4	0.2056
0.0811	77.4	0.1847
0.094	66.8	0.2661
0.1073	58.5	0.3344
0.1175	53.4	0.3037
0.125	50.2	0.2931
0.14	44.8	0.2274
0.1804	34.8	0.2689
0.211	29.7	0.2016
0.222	28.2	0.1821
0.324	19.3	0.2098
0.646	9.7	0.1529
0.678	9.2	0.1532
0.702	8.9	0.1525
0.725	8.6	0.1513



The El-Centro 1940 design response spectrum (Sa) is used for the three cases to calculate the vertical dynamic loads on the masses. Vertical dynamic loads on the masses were found after finite element dynamic analysis (CQC method), and they are shown in Table 2.4.

Table 2.4. Masses vertical dynamic loads

Case “1”					
Joint #	Load (Kg)	Joint #	Load (Kg)	Joint #	Load (Kg)
1	0.0	7	3446	13	4640
2	13250	8	1934	14	5015
3	406	9	261	15	3945
4	2986	10	27700	16	4267
5	4372	11	295	17	4870
6	4462	12	2751	18	0.0
Case “2”					
Joint #	Load (Kg)	Joint #	Load (Kg)	Joint #	Load (Kg)
1	3433	7	3830	13	4041
2	11820	8	2672	14	4506
3	567	9	336	15	3869
4	3311	10	26950	16	4251
5	3850	11	322	17	4550
6	3861	12	2492	18	0.0
Case “3”					
Joint #	Load (Kg)	Joint #	Load (Kg)	Joint #	Load (Kg)
1	18710	7	5683	13	2624
2	26000	8	3608	14	3703
3	638	9	350	15	3839
4	1942	10	27410	16	4216
5	2448	11	250	17	3960
6	5040	12	1303	18	0.0

Table 2.5 shows the modal joint (mass) vertical displacements and the bridge modal periods. Positive joint displacements represent the upward joint displacements.

Table 2.5. Joint displacement (mm) and the deck bridge modal periods

Joint	Modes (file: Hinge)				Modes (file: Hinge B)				Modes (file: Hinge C)			
	1	2	3	4	1	2	3	4	1	2	3	4
1	-0.16	-1.3	0.87	2.43	-0.16	-1.35	0.94	3.3	-0.16	1.43	1.07	2.42
3	0.05	0.4	-0.27	-0.75	0.05	0.41	-0.28	-0.81	0.05	-0.42	-0.28	-0.45
4	0.24	1.89	-1.21	-2.92	0.24	1.89	-1.17	-2.44	0.24	-1.85	-0.97	-0.66
5	0.39	2.89	-1.63	-2.37	0.40	2.86	-1.5	-1.36	0.40	-2.71	-1.02	0.69
6	0.48	3.15	-1.4	0.41	0.48	3.08	-1.2	1.14	0.49	-2.86	-0.55	2.12
7	0.48	2.64	-0.68	2.87	0.48	2.56	-0.49	2.85	0.48	-2.33	0.10	2.50
8	0.36	1.55	0.04	2.8	0.36	1.5	0.15	2.39	0.36	-1.13	0.50	1.60
9	0.09	0.29	0.15	0.58	0.09	0.28	0.16	0.45	0.09	-0.24	0.22	0.24
11	-0.1	-0.26	-0.23	-0.49	-0.1	-0.25	-0.24	-0.35	-0.10	0.22	-0.28	-0.15
12	-0.61	-0.99	-1.62	-1.34	-0.61	-0.93	-1.65	-0.79	-0.61	0.79	-1.73	-0.05
13	-1.24	-1.18	-2.83	-0.52	-1.23	-1.1	-2.85	-0.70	-1.23	0.90	-2.86	0.59
14	-1.95	-0.85	-2.95	0.91	-1.95	-0.77	-2.95	0.87	-1.95	0.59	-2.88	1.06
15	-2.72	-0.09	-1.66	1.45	-2.72	-0.05	-1.65	1.07	-2.72	-0.42	-1.55	0.86
16	-3.53	0.94	0.77	0.39	-3.52	0.92	0.79	0.20	-3.52	-0.88	0.85	-0.02
17	-4.34	2.08	3.75	-1.73	-4.33	2.0	3.77	-1.33	-4.33	-1.81	3.76	-1.28
18	-4.54	2.37	4.51	-2.31	-4.53	2.27	4.54	-1.75	-4.53	-2.04	4.5	-1.61
T(S)	3.386	0.672	0.397	0.182	3.387	0.688	0.401	0.204	3.390	0.733	0.415	0.273

Static analysis based on the vertical dynamic loads at the joints (Table 2.4) is used for all three cases, and final vertical reactions at J#18 (end of the long cantilever) are shown in Table 2.6.

Table 2.6. Joint #18 vertical load (Kg)

Case Number	Up lift force at joint# 18 (Kg)
1	12360
2	11600
3	10560

3. CONCLUSIONS

In this study the one tenth of the horizontal El-Centro 1940 accelerations have been applied as a vertical accelerations as an unknown earthquake. The first four modes of the bridge vibrations are considered in order to find the maximum deflections and up-lift forces. The maximum up-lift force (positive sign) at the end of the long cantilever (J#18) was about 12360Kg. The designed hinge dead load plus pre-stressing reaction at the end of the long cantilever was about 890100Kg that is about 72 times of the maximum dynamic up-lift force. Even with considering the El-Centro 1940 earthquake horizontal accelerations as vertical ground accelerations, the joint#18 dynamic up-lift force would be about 1/7.2 long cantilever reaction. Therefore there is no need for the vertical restrainers at hinge locations of this bridge. Figure 3.1 shows the mode #1 joint displacements and the dynamic up-lift reaction at J#18.

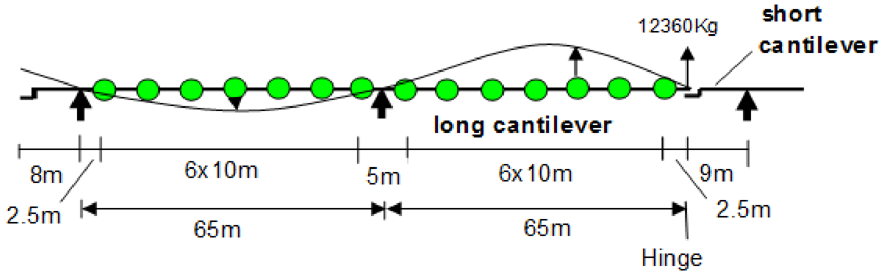


Figure 3.1. Bridge frame deflection and uplift force

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