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EFFECT OF UNSEATING PREVENTION CABLE FOR CONTINUOUS GIRDER BRIDGES

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ABSTRACT :

Unseating prevention system is important to protect bridges from falling due to huge earthquakes. However, the design method of the unseating prevention system is not established. Nevertheless, few researches have been done on the unseating prevention system for continuous girder bridges. This paper shows a rational design method of unseating prevention cables for continuous girder bridges. The results showed that the unseating prevention cable reduced the bending moment of the girder effectively at the support adjacent to the fallen end, but the effect is different by the cable stiffness. Japanese current design code specifies only the capacity load of the cables; however, the results showed that the stiffness of the cable should also be specified to prevent the collapse of the girder.

KEYWORDS: unseating prevention cable; continuous girder bridge; necessary cable stiffness; fall of girder; cable capacity

1. INTRODUCTION

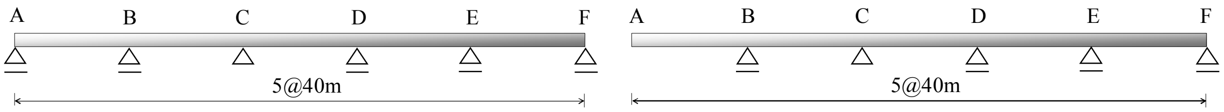
As many simply supported girder bridges collapsed and fell down in 1995 Kobe earthquake, the Design Specification for Japanese Highway Bridges (Japan Road Association, 2002a) specifies new unseating prevention system and its necessary capacity. Then the unseating prevention cables were set to all the bridges, and it was studied by many researchers up to today (Shimanoe 2000, Kawashima 2001, 2002, Abe 2004, Izuno 2004, etc.). After Kobe earthquake, some of the old simply supported girder bridges were tied to the adjacent girders to be the continuous girder bridges. Furthermore, most of the new bridges are designed as continuous girder bridges. However, few researches have been done on the unseating prevention system for continuous girder bridges.

As a simply supported girder bridge is statically determinate structure, the girder will fall if a support of one end of bridge breaks. On the other hand, as a continuous girder bridge is statically indeterminate structure, even if one support breaks, it won't fall if a girder doesn’t break. But a continuous girder bridge is possible to fall if the section force exceeds the yield limit in addition to the break of the support. It means that the mechanism of the girder falling differs for the continuous girder bridge and simply supported girder bridge. However the same design code for the unseating prevention cables is applied for any kind of bridges: it is based on the reaction force for the dead load at the girder end.

This paper deals with the necessary capacity and stiffness of the unseating prevention cables for continuous girder bridges.

2. EFFECT OF UNSEATING PREVENTION CABLE ON FALLING GIRDER

When a girder falls from a pier, the section force at some parts of the girder increases intensively. This chapter



(a) 5-span continuous girder bridge model (b) Collapsed support model

Figure 1 Analytical models

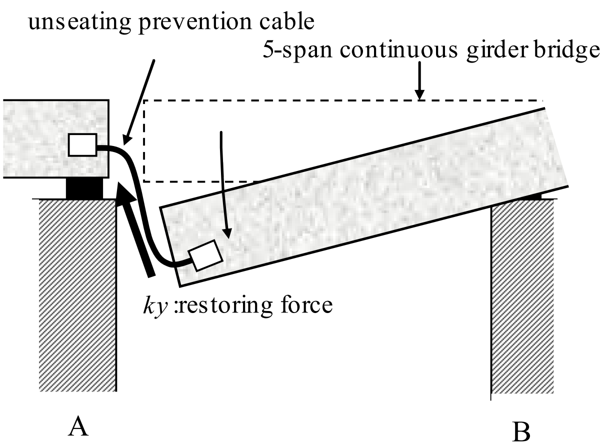


Figure 2 Mechanism of unseating prevention cable

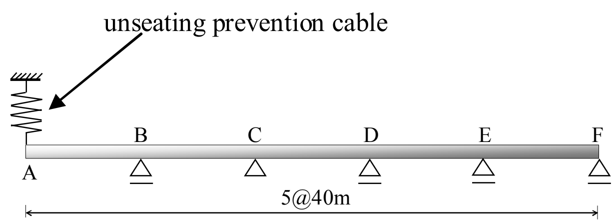


Figure 3 Collapsed support model with unseating prevention cable

describes the influence of the girder falling on the section force distribution with and without the unseating prevention cable.

2.1. Analytical Models

An analytical model used is a 5-span continuous girder bridge with its dead load of 154kN/m as shown in Figure 1(a). The supports are named as A, B, ..., F from the left. The left most point A is supposed to collapse as shown in Figure 1(b). The unseating prevention cable is installed between point A and the neighboring girder or abutment, which is assumed as a fix point. When the left end of the girder (point A) falls from its support, the unseating prevention cable pulls up the falling girder with the restoring force ky (k: cable stiffness, y: fallen displacement) as shown in Figure 2. Figure 3 shows the collapsed support model with the unseating prevention cable at point A.

2.2. Effect of Unseating Prevention Cable

Figure 4 shows the effect of the unseating prevention cable on the bending moment distribution. The horizontal axis shows the place of the supports, and the vertical axis shows the bending moment at each point of the girder. The bending moment at point B before falling is -20MN.m, but the bending moment of the falling girder is 6 times larger. The bending moment at point C of the falling girder becomes positive, though which is negative before falling. After setting the unseating prevention cable, the bending moment at point B decreases to about 10% to 80%, and the bending moment at point C remains negative.

Table 1 shows the reaction forces at point B and C for each case. In which, k, F, and R_B, ..., R_F are the cable stiffness, reaction force of the cable, and reaction forces at point B to F, respectively. The cable stiffness of infinity (∞) corresponds to the original 5-span continuous girder bridge as shown in Figure 1(a). When the

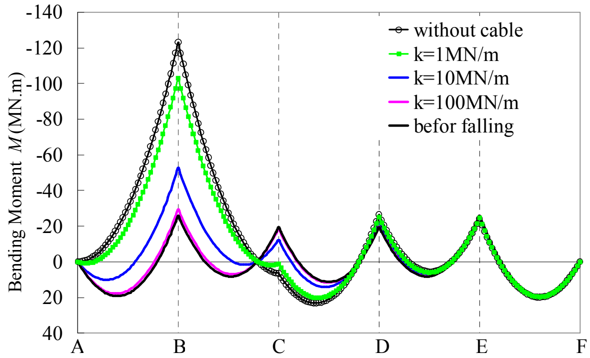


Figure 4 Bending moment distribution

Table 1 Reaction forces at supports and cable

stifness of cable	reaction force of cable	raction force at support(MN)				
k	F(MN)	R_B	R_C	R_D	R_E	R_F
without cable	-	12.48	2.09	7.04	6.71	2.47
1MN/m	0.51	11.33	2.90	6.82	6.76	2.47
10MN/m	1.76	8.49	4.92	6.28	6.90	2.44
100MN/m	2.34	7.17	5.85	6.03	6.96	2.43
∞	-	6.97	6.00	6.00	6.97	2.43

unseating prevention cable isn't installed, the reaction force at point B is 2 times larger than the 5-span continuous girder bridge. The reaction force at point B with unseating prevention cable decreases to 50% at the maximum.

3. DESIGN OF UNSEATING PREVENTION CABLES FOR CONTINUOUS GIRDER BRIDGE

This chapter proposes the design method of unseating prevention cables for continuous girder bridges according to the dynamic analysis.

3.1. Analytical Models

Analytical models used are as same as Chapter 2, shown in Figures 1 and 2. The only load which acts during falling is assumed as dead load, and no other external force such as earthquake inertia force is considered. Free falling is analyzed numerically considering the gravity. The time increment of dynamic analysis is set to 0.001 second, and the damping factor of girder is assumed as 0.02.

As the yield position of variable cross section girder bridges would differ from that of constant cross section bridges, variable cross section girders are considered in this chapter as well as constant cross section girders. The cross section of the girder is designed according to the design bending moment specified by the Design Specification for Japanese Highway Bridges (Japan Road Association, 2002b), shown in Figurer 5. The design lengths of the sections are shown in Table 2. Cases 1 and 2 are variable cross section girder, and case 3 is constant cross section girder. The moment of inertia of the section of the constant section girder is then calculated according to the moment ratio of Eqn. 3.1.

$$I_i = I_B \frac{M_i}{M_B} \tag{3.1}$$

whereas, M_B is the bending moment at point B, M_i is the design bending moment at section-i. I_B is the moment of inertia at point B, and I_i is the moment of inertia at section-i. Table 2 summarizes the section characteristics for each case.

The hysteretic curve for the moment M and the curvature φ of the girder is modeled using a nonlinear tri-linear model as shown in Figure 6. The 100% line in the figure corresponds to the design bending moment at each section. For steel girder bridges, the Design Specification for Japanese Highway Bridges gives safety factor of 1.7 to yield of the girder. Accordingly, this study assumes that the girder yields at 170% of design bending moment, and the rigidity after yielding decreases to 1/100 of the initial rigidity. Similarly, the negative flexural

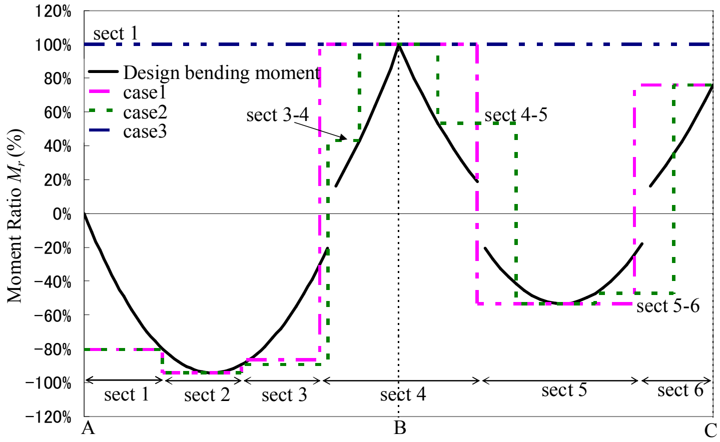


Figure 5 Design bending moment

Table 2 Design of girder cross section

(a) case 1

section number	l_i (m)	M_i / M_B
sect 1	0-10	0.80
sect 2	10-20	0.94
sect 3	20-30	0.89
sect 4	30-50	1.00
sect 5	50-70	0.53
sect 6	70-90	0.76

(c) case 3

section number	l_i (m)	M_i / M_B
sect 1	0-200	1.00

(b) case 2

section number	l_i (m)	M_i / M_B
sect 1	0-10	0.80
sect 2	10-20	0.94
sect 3	20-31	0.89
sect 3-4	31-35	0.43
sect 4	35-45	1.00
sect 4-5	45-55	0.53
sect 5	55-65	0.53
sect 5-6	65-75	0.47
sect 6	75-85	0.76

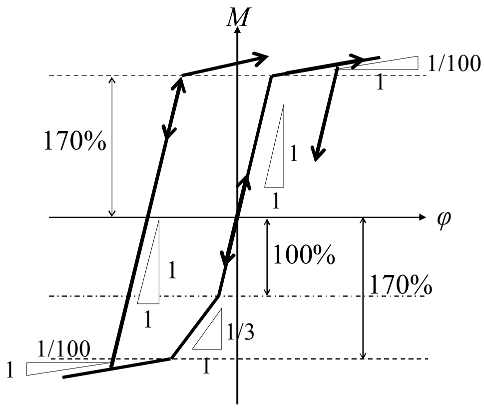


Figure 6 Hysteretic model of girder

rigidity decreases to 1/3 of the initial rigidity at design bending moment (the first yields point), and decreases to 1/100 of the initial rigidity at 170% of the design bending moment (the second yields point). Decrease ratio of 1/3 and 1/100 correspond to crush of the slab and yield of the girder, respectively.

3.2. Results from Dynamic Analysis

3.2.1 Yield condition of girder

Figure 7 shows distribution of the bending moment at 0.2 seconds after falling. The horizontal axis shows the place between point A and point B, and the vertical axis shows the ratio of the bending moment to the design bending moment. The design bending moment for this region is negative. The broken line in Figure 7 corresponds to the first yield point level (M_r is 100%).

In the variable cross section girders (cases 1 and 2), the moment ratio exceeds M_r=100% at the adjacent cross section to point B. On the contrary, in the constant cross section girder (case 3), the girder yields at point B.

3.2.2 Time history response of girder

Figure 8 shows the bending moment-time histories at point B. A chain line in Figure 8 shows the level of 170% of the design bending moment. The × marks show the bending moment at any cross section reached 170% of the design bending moment. In the cases except the constant cross section girder of case 3, the design bending

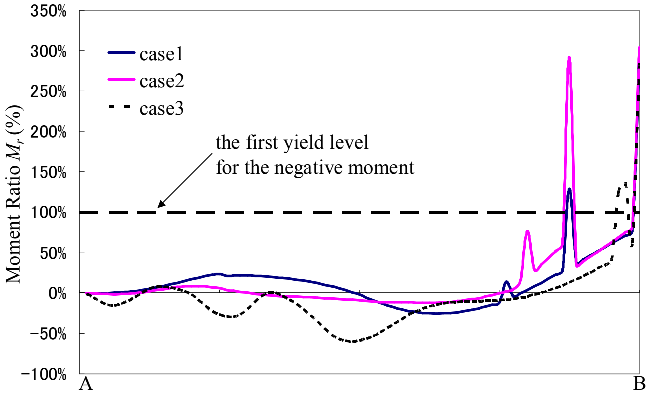


Figure 7 Distribution of bending moment

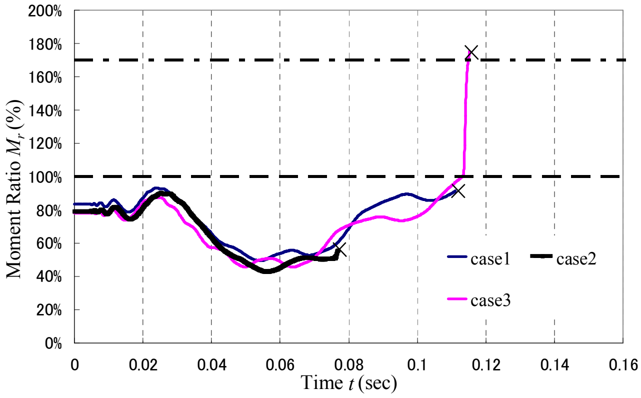


Figure 8 Bending moment-time histories at point B

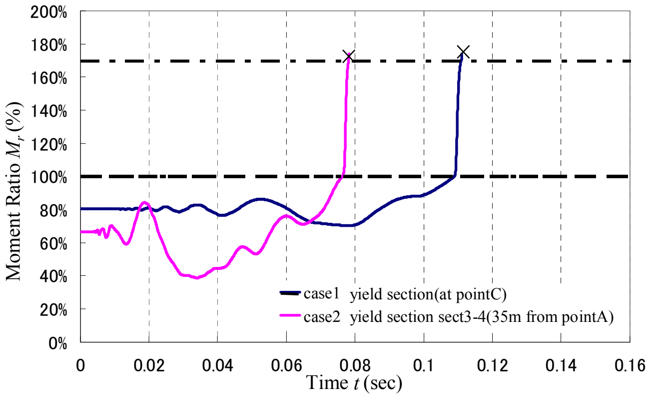


Figure 9 Bending moment-time histories for cases 1 and 2

moment at another section reached 170% level prior to point B. Figure 9 shows the bending moment-time histories for cases 1 and 2 at the element which yields first. The girder vibrates for a few seconds immediately after girder falling and falls continuously after 0.03 seconds.

3.3. Effect of Unseating Prevention Cable

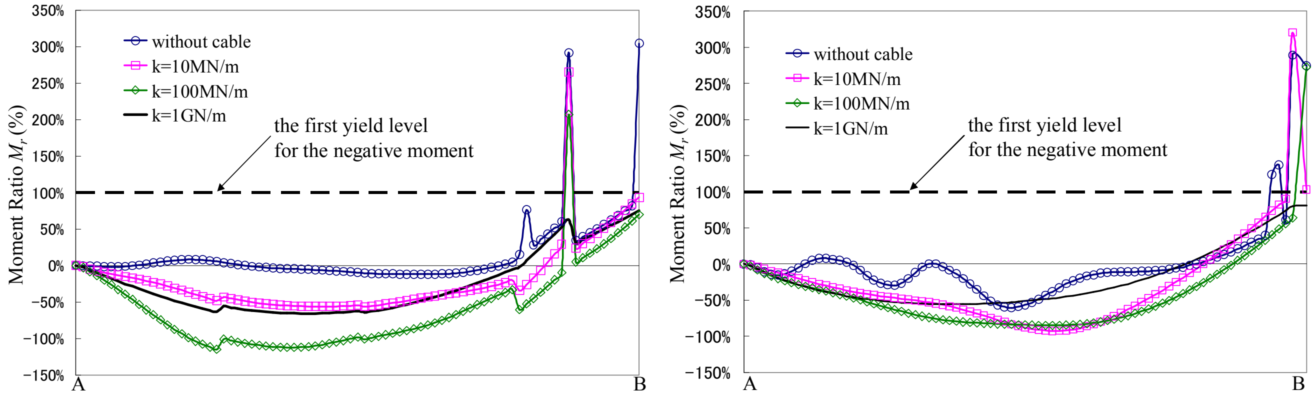
3.3.1 Distribution of bending moment

Figure 10 shows distribution of bending moment between point A and point B at 0.2 seconds after falling. The results for the cable stiffness of 10MN/m, 100MN/m, 1GN/m, and without cable cases are plotted. The bending moment reaches 100% for both cases 2 and 3 if the cable stiffness is less than 1GN/m, while the cable stiffness of 1GN/m results in the elastic response.

3.3.2 Time history response of girder

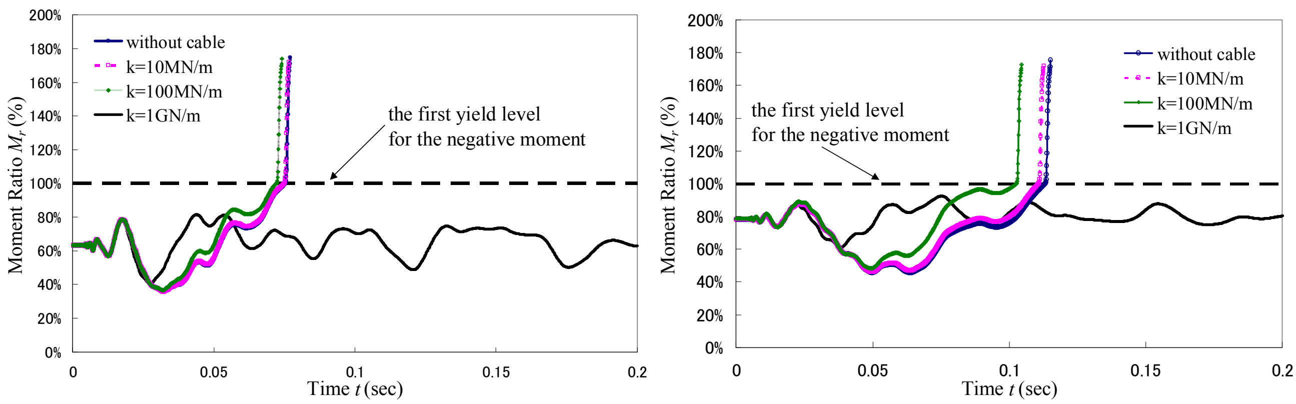
Figure 11 shows the bending moment-time histories at the element which yields first (35m from point A for case 2, point B for case 3). Unseating prevention cable of other than 1GN/m has no effect on the girder response. Moreover, when the bending moment becomes 100% of the design moment, it shows abrupt increase over 170%.

The variable cross section girder of case 2 yields faster than the constant cross section girder of case 3, because



(a) case2 (b) case3

Figure 10 Distribution of bending moment with and without unseating prevention cable



(a) case2 (b) case3

Figure 11 Bending moment-time histories with and without unseating prevention cable

it yields at the weaker section than the constant section girder.

3.4. Necessary Stiffness of Cable

Figure 12 shows necessary cable stiffness to prevent yield of the girder. The horizontal axis shows the stiffness ratio of the cable, and the vertical axis shows the rate of change of the bending moment. The stiffness ratio of the cable is the dimensionless parameter: the ratio of the cable stiffness k to the flexural rigidity of the girder EI and the span l as shown in Eqn.3.2. β=0 corresponds to the case without a cable.

$$\beta = \frac{k}{EI/l^3} \tag{3.2}$$

The flexural rigidity in the variable cross section girder is chosen as the section at point B, and the span length is chosen between point A and point B. The bending moment becomes 1000% without a cable, while it decreased to 300% with the cable. The bending moment decreases abruptly as β becomes larger than the particular value, and shows elastic response. The smallest β to keep the girder elastic is 220 for the variable cross section girder of case 2, and 270 for the constant cross section girder of case 3.

If β is more than 220 (case 2) or 270 (case 3), the bending moment becomes less than 170%. The necessary

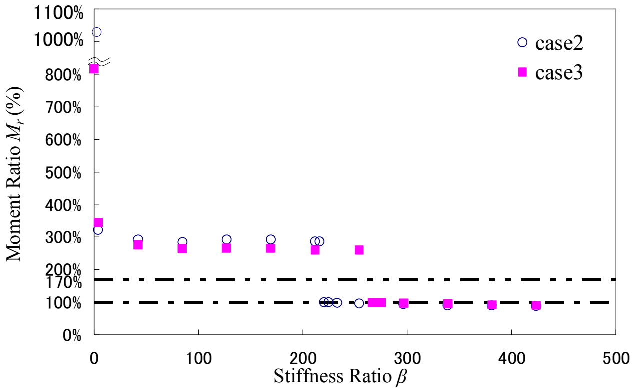


Figure 12 Necessary stiffness ratio of cable

cable length is solved by the span length and flexural rigidity, but combination is infinite. Therefore, cable length is calculated in this section from the natural frequency of the girder.

The first natural frequency of a Bernoulli-Euler beam, with the dead load q per unit length is described as Eqn. 3.3.

$$f_0 = \frac{\pi}{2{l_1}^2}\sqrt{\frac{EIg}{q}} \tag{3.3}$$

whereas, g is gravity acceleration, l_1 is the span length between points A and B. The reaction force at point A is calculated by Eqn. 3.4.

$$R_A = \frac{15}{38} q l_1 \tag{3.4}$$

On the other hand, the necessary sectional area A of the cable according to the current design method is shown in Eqn. 3.5. Japanese Highway Bridge Code specifies that the cable must have the capacity of 1.5 times larger than the reaction force at the end support.

$$A = \frac{1.5 R_A}{\sigma_y} = \frac{45 q l_1}{76 \sigma_y} \tag{3.5}$$

whereas, R_A is the reaction force at point A, and σ_y is the yield stress of the girder. The cable stiffness k is derived from the Young's modulus E_s and the cable length L as shown in Eqn. 3.6.

$$k = \frac{E_s A}{L} = \frac{45 E_s q l_1}{76 \sigma_y L} \tag{3.6}$$

The stiffness ratio of the cable defined by Eqn. 3.2 is expressed as Eqn. 3.7 from Eqns. 3.3 and 3.6.

$$\beta = \frac{45 \pi^2 E_s g}{304 \sigma_y {f_0}^2 L} \tag{3.7}$$

The necessary cable length for β_a as the minimum value of β is then derived by Eqn. 3.8.

$$L \le \frac{45\pi^2 E_s g}{304\sigma_y f_0^{\,2}\beta_a} \tag{3.8}$$

For an example, if we use the Young's modulus of cable as 2GPa, yield stress of cable as 1.2GPa, and the natural frequency of the girder as 3Hz, the maximum cable length corresponding to β=220 is 0.9m, and β=270 is 0.6m from Eqn. 3.8. They are too short compared to the usual length of the cable of 2m to 10m (β=132 to 26.5). This was caused by rather small sectional area specified by the current code of Eqn. 3.5. This result shows the necessity of the design code considering both the capacity and the stiffness of the unseating prevention cable.

4. CONCLUSIONS

This paper studied the influence of unseating prevention cable on falling of continuous girder bridge. The major results obtained in this study are as follows.

1） The unseating prevention cable reduced the bending moment of the girder effectively at the support adjacent to the fallen end, but the effect is different by the cable stiffness.

2） A section at the support adjacent to the fallen end yielded first in the constant cross section girder, while a section between supports yielded first in the variable cross section girder.

3） The necessary stiffness of unseating prevention cables for the constant cross section girder was larger than that for the variable cross section girder. The necessary stiffness ratio of the cable to prevent yield of the girder was able to be determined by the cable stiffness, the flexural rigidity of the girder and the span length. The cable should have enough stiffness as well as enough capacity for the unseating prevention system of the continuous girder bridges.

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