Investigation of earthquake resistance and cost effect on hybrid cable-stayed bridge with two girders

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Abstract

Nowadays hybrid structures composed of concrete and steel are widely constructed in Japan and known to be superior against earthquakes and to be cost effective. In this study, a bridge of 340m in length which crosses a valley was used as a model object, in which the hybrid cable-stayed bridge of two girders was proposed as the most economical bridge, having significant resistance against earthquake. After having carried out investigations from many different aspects, this paper focuses on the subjects as shown below.

By imposing bearing support condition on the top of pier (for example, fixed bearing, movable bearing, base isolation bearing) as parameters, dynamic analyses were carried out by using 3-dimensional analytical models that introduce inelastic elements in a RC pier. As a result, the optimum dimension of the RC pier and bearing conditions were determined. To investigate the effects on the isolation bearing, parametric analysis on characteristics of the isolation bearing was carried out. In addition to a design for a hybrid cable-stayed bridge with two girders, a PC continuous rigid frame bridge and steel arch bridge were designed to investigate cost effectiveness. By calculating quantities of works, cost comparisons of these bridges was carried out. It was concluded from these investigations that by using the isolation bearing the proposed bridge was superior in cost effectiveness and earthquake resistance.

1 Introduction

The bridge plan has a great deal of influence on the cost performance of the bridge construction. The combination of bridge type and span is important for designing the bridge. If a bridge is constructed over a deep valley, the
piers would be constructed at middle of the valley, then a PC continuous rigid frame bridge would be selected generally from the economical requirement. On the contrary, if the pier can't be constructed by various reasons such as geometrical constrains over the valley, arch bridge is constructed. Nowadays, hybrid structures composed of concrete and steel are widely constructed in Japan. For example, two edge steel girders as representative of rational girder, and hybrid rigid frame bridges whose girders are made by steel and whose piers are made by reinforced concrete (RC) is constructed. By considering these practical situations, we proposed a 3-continuous hybrid cable-stayed bridge with two girders (abbreviated as "hybrid bridge") from the point of earthquake resistance and cost effectiveness. This study firstly proposed "hybrid bridge" based on the assumption that it would possess advantageous characteristics against earthquake and cost effectiveness. Secondary the study analyzed the validity of assumption by dynamic analysis of the structure under earthquake conditions and the economical comparison with other competitive bridges.

2 The bridge outline

The proposed bridge has the characteristics as described below: Center span length - 170m, height of tower - 74m, pier - made of RC, main girder - two edge steel girder. Figure 1 shows general view. Superstructure has two edge girders which have PC slab, and piers and towers are made of RC. The width of piers widens as it goes to the lower part. Figure 2 shows the cross section of the superstructure and the pier.

![Figure 1. General view](image-url)
3 Analytical model and condition

3.1 Analytical model
Analytical model is shown in Figure 3. The main girders are replaced by two beams having transverse rigid bars. The center beam was imposed having moment of inertia and polar moment of inertia. The main tower and the pier in the model similar to the actual shape by considering three dimensional girders. The boundary conditions are imposed to constrain the vertical movement of main girder, and restricted the transverse displacement and torsional rotation as well, but other parts remained to move freely. The foundation of the pier is fixed and the end of cables are jointed by the pin.

3.2 Bearing condition
The seismic isolators are installed on the top of pier. In this study, lead rubber bearing (LRB) was used as the seismic isolator. On the horizontal spring in longitudinal and transverse directions have imposed bilinear restoring force characteristics. The characteristics of seismic isolators are shown in table 2. Three types of bearing conditions are used in this study. Bearing condition of case 1 is fixed bearing, case 2 is movable bearing, base isolation is used for case 31~case 51. Lead rubber bearing is used as the isolation bearings. Qd (yield load of the isolation bearing). K1 (Primary
K2 (secondary stiffness) are considered as parameters of the isolation bearing. Qd, K1 and K2 have high interrelated relations with area of bearing support and shearing modulus. These relationships are calculated by formula (1.1)–(1.8). Once dimension of base isolation and shearing modulus are fixed, Qd, K1, K2 are calculated by the formulae. In this study, the areas of base isolation are assumed as 1.0 m², 1.5 m², 2.0 m². Further, shearing modulus is 0.8 N/mm², 1.0 N/mm², 1.2 N/mm². Ratio of area of lead to area of the isolation is 6%. Under these conditions, parametric studies are carried out.

\[
K_B = \frac{A_R \cdot G \cdot \gamma + A_p \cdot q}{U_{BE}} \quad \ldots (1.1)
\]

\[
q = a_0 + a_1 \cdot \gamma + a_2 \cdot \gamma^2 \quad \ldots (1.2)
\]

\[
\gamma = \frac{U_{BE}}{\Sigma \tau_e} \quad \ldots (1.3)
\]

\[
h_B = \frac{-2Q_d(U_{BE} + Qd/(K_2 - K_1))}{\pi \cdot U_{BE}(Qd + U_{BE} \cdot K_2)} \quad \ldots (1.4)
\]

\[
Q_d = A_p \cdot q_0 \quad \ldots (1.5)
\]

\[
a_0 = b_0 + b_1 \cdot \gamma \quad \ldots (1.6)
\]

\[
K_1 = 6.5 \cdot K_2 \quad \ldots (1.7)
\]

\[
F = A_R \cdot G \cdot \gamma + A_p \cdot q \quad \ldots (1.8)
\]

**Table 2. Bearing condition**

<table>
<thead>
<tr>
<th>Case</th>
<th>AR (m²)</th>
<th>AP (m²)</th>
<th>Qd (kN)</th>
<th>K1 (kN/m)</th>
<th>K2 (kN/m)</th>
<th>KB (kN/m)</th>
<th>hB</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fixed bearing</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Case 1</td>
<td>-</td>
<td>-</td>
<td>∞</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Movable bearing</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Case 2</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>The isolation bearing</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LRB(1)</td>
<td>Case 31</td>
<td>1.0</td>
<td>0.6</td>
<td>510</td>
<td>11943</td>
<td>1837</td>
<td>3285</td>
</tr>
<tr>
<td>Case 32</td>
<td>1.5</td>
<td>0.9</td>
<td>765</td>
<td>13652</td>
<td>2100</td>
<td>5025</td>
<td>0.249</td>
</tr>
<tr>
<td>Case 33</td>
<td>2.0</td>
<td>1.2</td>
<td>1020</td>
<td>25076</td>
<td>3858</td>
<td>9075</td>
<td>0.265</td>
</tr>
<tr>
<td>The isolation bearing</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LRB(2)</td>
<td>Case 41</td>
<td>1.0</td>
<td>0.6</td>
<td>510</td>
<td>15658</td>
<td>2409</td>
<td>3886</td>
</tr>
<tr>
<td>Case 42</td>
<td>1.5</td>
<td>0.9</td>
<td>765</td>
<td>18201</td>
<td>2800</td>
<td>6331</td>
<td>0.233</td>
</tr>
<tr>
<td>Case 43</td>
<td>2.0</td>
<td>1.2</td>
<td>1020</td>
<td>30329</td>
<td>4666</td>
<td>11252</td>
<td>0.264</td>
</tr>
<tr>
<td>The isolation bearing</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LRB(3)</td>
<td>Case 51</td>
<td>1.0</td>
<td>0.6</td>
<td>510</td>
<td>18489</td>
<td>2845</td>
<td>4485</td>
</tr>
<tr>
<td>Case 52</td>
<td>1.5</td>
<td>0.9</td>
<td>765</td>
<td>27136</td>
<td>4175</td>
<td>8021</td>
<td>0.225</td>
</tr>
<tr>
<td>Case 53</td>
<td>2.0</td>
<td>1.2</td>
<td>1020</td>
<td>95558</td>
<td>14701</td>
<td>12192</td>
<td>0.255</td>
</tr>
</tbody>
</table>

**Notes:**
- K_B: Equivalent stiffness of the isolation bearing
- h_B: Equivalent damping constant of the isolation bearing
- U_{BE}: Effective design displacement of the isolation structure
- Q_d: Yield load of the isolation structure
- K_1, K_2: Primary and secondary stiffness of the isolation bearing
- A_R: Area of the isolation bearing
- A_p: Area of the lead
- G: Shearing modulus of gum
- \Sigma \tau_e: Total thickness of gum
3.3 Non-linear characteristics of members

Non-linear characteristics of the pier are evaluated in each cross section as shown in Figure 2. Skelton of moment-curvature relationship was modeled by tri-linear having two breaks at crack and yield points. Skelton curve at the base of pier is shown in Figure 4. Takeda model is adopted for determining restoring force characteristics. Moment-curvature (M-Ø) relationship in skelton curve of pier is decided by each cross-section and initial axial force obtained by self weight analysis.

Regarding both pier, 'crack-yield-ultimate' in M-Ø relationship is decided using stress-strain curve in specification of highway bridge 1996.

3.4 Analytical condition

Newmark' βmethod (β=1/4) in used for integration, and time interval is 0.001 seconds. Based on conventional practice, the damping constant is assumed to be 2% in superstructure and pier, 5% in tower, and 1% in cable. The damping constant of foundation is ignored. As an input acceleration time history waveform, such as Type 1 (large scale plate boundary earthquake) and Type 2 (the inland direct strike type earthquake) earthquake motions are used. Input seismic waveforms are shown in Fig. 5. These are fitted for the acceleration response spectrum defined in the specification of highway bridges in Japan. The direction of seismic force is longitudinal direction of bridge. Ground class is Class 1 which has good diluvial ground or rock mass.

4 The Eigen value analysis and damping

This bridge is supposed to have complicate response property by the different height of piers etc, so that it is very important to understand the vibration characteristics. The result of the eigen value analysis and effective mass are shown in Table 3. and the shapes of the primary mode in case of
fixed bearing, movable bearing, isolation bearing are shown in Figure 6. Based on these results, it is clear that the higher order mode plays important role such that it can not be ignored for obtaining accurate result. Rayleigh damping constants are determined by using two modes. Definition of the Rayleigh damping constants shows in Figure 7. It is clear that the higher the frequency, the higher the damping constants.

Table 3. Result of eigen value analysis

<table>
<thead>
<tr>
<th>Bearing Type</th>
<th>Fixed bearing</th>
<th>Movable bearing</th>
<th>LRB(CASE51)</th>
</tr>
</thead>
<tbody>
<tr>
<td>The degree</td>
<td>Period (sec)</td>
<td>The effective mass ratio(%)</td>
<td>Period (sec)</td>
</tr>
<tr>
<td>1</td>
<td>2.463</td>
<td>65</td>
<td>4.974</td>
</tr>
<tr>
<td>2</td>
<td>2.357</td>
<td>0</td>
<td>2.814</td>
</tr>
<tr>
<td>3</td>
<td>1.498</td>
<td>8</td>
<td>1.975</td>
</tr>
<tr>
<td>4</td>
<td>1.031</td>
<td>0</td>
<td>1.152</td>
</tr>
<tr>
<td>5</td>
<td>0.915</td>
<td>0</td>
<td>0.971</td>
</tr>
</tbody>
</table>

Figure 6. The mode figure (Primary)

Figure 7. Rayleigh damping constants

Figure 8.1. Fixed bearing

Figure 8.2. Movable bearing

Figure 8.3. The isolation bearing
5 Results of analyses

5.1 Bending moment and displacement of the pier and tower

Figures 8 shows the moment-curvature hysteresis loop at the base of the pier in the longitudinal direction. To investigate the effect of the isolation bearing, maximum bending moment at the base of pier is shown Figure 9. Maximum moment is different between Type 1 and Type 2. Since this bridge has long eigenperiod, the maximum moment of the pier occurs in the case of Type 1 waveform in comparison with that of Type 2. Based on Figure 9, it is clear that regardless of the characteristics of the isolation bearings, maximum moment in the base of pier decreases to 15% to 20% of the fixed bearing. Zero on horizontal axis represents movable bearing, \( \infty \) represents fixed bearing. On the other hand, the bending moment at the base of tower in Type 1 is bigger than Type 2. It is clear that if the shearing modulus is selected appropriately, the bending moment at the base of the tower will be definitely reduced.

![Figure 9. Comparison of maximum moment](image)

5.2 Displacement of bearing and girder

Figure 10 shows maximum displacement at the girder end, top of the pier and the isolation bearing respectively. Compared with the result of Type 2 and Type 1, it is clear that maximum displacement at the isolation bearing, girder and top of the pier occur in Type 1 in comparison with that of Type 2.

Maximum displacement at the isolation bearing is influenced by both Qd and shearing modulus at the Type 1 waveform in comparison with Type 2. The bigger the shearing modulus, the smaller the maximum displacement of the bearing become. Maximum displacement of the girder end is influenced by Qd at Type 1 waveform. When the Qd is 1020kN, it is almost identical to that of fixed bearing.
Regardless of the parameter Qd, maximum displacement at the top of the pier is constant, that value is small in comparison with the fixed and movable bearing. Figure 11 shows hysteresis loop of displacement at the isolation bearings.

![Figure 10.1. girder end](image1)

![Figure 10.2. top of pier](image2)

![Figure 10.3. the bearing](image3)

![Figure 11. Hysteresis loop displacement at the bearing (case 21)](image4)

![Figure 12. Horizontal force](image5)

1. isolation bearing

2. the tower
5.3 Horizontal force induced to the isolation bearing and the tower

Horizontal forces induced to the isolation bearing and the tower is shown in Figure 12. The contribution of horizontal force varies with the characteristics of the individual bearings. The bigger the stiffness of the isolation bearing becomes, the horizontal force of the isolation bearing increases. The horizontal force is induced completely from the tower in case of movable bearing, on contrary horizontal force is small in case of fixed bearing, but was not negligible. This is considered as a reason why self-weight of the tower is influenced to the horizontal force. Since seismic load is damped by using the isolation bearing, it is clear that horizontal force decreases compared to using fixed bearing.

6 Cost effectiveness

To investigate cost effectiveness of “hybrid bridge”, PC continuous rigid frame bridge and Arch bridge are designed and compared. Figure 13 shows the general view of these model bridges for cost analyses. Table 4 shows the comparison of quantities which are obtained by designing each type of bridge. Cost was calculated according to a Japanese typical cost estimation guide. Table 5 shows the comparison of the cost. From this table, it is clear that “hybrid bridge” we proposed is superior to other bridge in cost effectiveness.
Table 4. Comparison of quantity

<table>
<thead>
<tr>
<th>Super structure</th>
<th>Bridge type</th>
<th>hybrid bridge</th>
<th>PC rigid frame bridge</th>
<th>arch bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Span length</td>
<td>85.0+170.0+85.0m</td>
<td>90.0+160.0+90.0m</td>
<td>260m (arch span)</td>
</tr>
<tr>
<td>Girder type</td>
<td>2 edge girder</td>
<td>1-box girder</td>
<td></td>
<td>arch rib + stiffening girder</td>
</tr>
<tr>
<td>Girder height</td>
<td>H=1.8m</td>
<td>H=4.0~10.0m</td>
<td>H=2.8m</td>
<td></td>
</tr>
<tr>
<td>Material</td>
<td>Steel 748t</td>
<td>Concrete 4,490m³</td>
<td></td>
<td>Steel 2994t</td>
</tr>
<tr>
<td>Pier</td>
<td>Material</td>
<td>2,414m³ (P1) 2,284m³ (P2)</td>
<td>2,010m³ (P1) 1,800m³ (P2)</td>
<td>-</td>
</tr>
<tr>
<td>Foundation</td>
<td>Caisson type pile</td>
<td>Caisson type pile</td>
<td>Caisson type pile</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Dimension</td>
<td>φ=11m D=15m(P1),25m(P2)</td>
<td>φ=15m D=20m(P1),25m(P2)</td>
<td>φ=6.0m D=21.0m (P1, P2)</td>
</tr>
<tr>
<td></td>
<td>Material</td>
<td>1,806m³ (P1) 2,377m³ (P2)</td>
<td>3,533m³ (P1) 4,416m³ (P2)</td>
<td>1,190m³ (P1, P2)</td>
</tr>
<tr>
<td>Erection</td>
<td>Cable crane</td>
<td>Cantilever</td>
<td>Cable crane</td>
<td></td>
</tr>
</tbody>
</table>

Table 5. Comparison of cost

<table>
<thead>
<tr>
<th>Bridge type</th>
<th>hybrid bridge</th>
<th>PC rigid frame bridge</th>
<th>arch bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Superstructure</td>
<td>1.00</td>
<td>0.94</td>
<td>2.42</td>
</tr>
<tr>
<td>Substructure</td>
<td>1.00</td>
<td>1.08</td>
<td>0.48</td>
</tr>
<tr>
<td>Total</td>
<td>1.00</td>
<td>1.01</td>
<td>1.42</td>
</tr>
</tbody>
</table>

7 Conclusions

Hybrid cable-stayed bridge with two girders was proposed and that outline of the structure was investigated. The maximum moment at the base of the pier and tower was reduced by using the base isolations on the top of the pier. PC continuous rigid frame bridge and arch bridge was designed under same conditions. Though "hybrid bridge" isn’t constructed in Japan yet, but it was concluded from the above investigations that this type of bridge was considered superior to the other in cost effectiveness and seismic resistance.

8 References

2] Suzuki, Tsuchida. A propose of 3-continuous hybrid cable stayed bridge. Proc. of the 55th annual Conf. of JSCE.