Behavior of a multiple spans cable-stayed bridge

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ABSTRACT: Bridges with multiple cable-stayed spans are more and more designed for large crossing projects, but the behavior of such bridges is by far different from the behavior of a standard cable-stayed bridge, with additional problems of stiffness in the middle spans and thermal expansion of the deck. We got the opportunity to participate in the design check of a five towers cable-stayed bridge with 300 meters spans and we examined the configuration between type of connection, stiffness of deck, stiffness of piers and pylons, in order to confirm the minimal structural cost. We analyzed on a FEM model the differences in the forces distribution following the connection case and we organized further calculations about the relationship between stiffness of deck, pylons and piers. Results are presented with particular focus about the impacts of asymmetric loading and thermal expansion of the deck on this multiple spans structure.

1 INTRODUCTION

1.1 Background of the study

Following the construction of the Millau Viaduct in France and the Rion-Antirion Bridge in Greece, multiple cable-stayed spans bridges are becoming popular as they appear to be aesthetic and as the reduction of the span length allow reduction of the towers height and load. This type of structure is more and more designed for large crossing projects and Ingerosec had already some occasion to propose this type of structure as alternative for some bridges in Japan.

Through the many technical discussions we had with Jean Schmitt of Ingerop when he was working with us in Tokyo, we could acquire some experience about the behavior of this type of structure. M. Virlogeux, adviser of Ingerosec Corp., presented also some specific problems of bridges with multiple cable-stayed spans in an interesting report published in SEI in 2001.

We had recently the occasion to bring technical assistance to the Hanoi University of Civil Engineering for the design check of a multiple cable-stayed span bridge and at this occasion we undertook a comparison between different configurations of the bridge as we were not convinced that the solution proposed by the designers was the most efficient. We want to present in this paper the main results of this comparison that enlighten the formula given in the previously referred report.

1.2 Aim of the study

One of the main problems of bridges with multiple cable-stayed spans is the behavior under live loads, as the deflections and bending moments in the deck are more influenced by the stiffness of the pylons and by the connection between deck and pylon than for a standard cable-stayed bridge. The second problem is the effect of deck length variation due to temperature and concrete creep and shrinkage.

The study hereafter will thus be concentrated on behavior under live loads and under length variation. For these loading cases, we will examine the effect of connection, of deck rigidity, of tower rigidity and of soil conditions. As we were checking a project, the basis data was the project data, presented hereafter, that will be used as reference for the comparison.

2 DESCRIPTION OF THE STUDIED BRIDGE

2.1 General arrangement of the bridge

As shown on fig. 1, the studied bridge is a 6 spans (5 pylons) with side spans lengths 148.5 m and main spans lengths 300 m. The horizontal alignment is straight and the vertical alignment is supposed horizontal for this study. The height of the towers is 74 m and the height of the piers is 37.60 m. The deck is a composite structure with two steel I-girders as edge beams joined by multiple crossbeams and a concrete top slab. The total height of the deck is 3.60 m for a width of 35.60 m.
The deck is suspended to the tower by 2 stay-planes with 11 cables per half-plane. The sizes of the cables vary from 60 to 170 cm². The space between cables is 12 m at deck level (from 24 m till 144 m) and 2 m at tower head (from +49 m till +69 m).

The original proposed connection between the deck and the pylon is a simply supported one, with one line of rubber bearings.

The piers foundations are steel pipe sheet piles foundation type. This type of foundation is well used in Japan, but the stiffness in rotation is smaller than for a standard piled foundation and is very sensitive to the lateral soil resistance.

2.2 Average height of the cables at tower

From the geometry of the cable stays, we can calculate the average height \( h \) above deck level of the cables at tower: \( h = 61 \) m. In accordance with the formula given in the report (see Reference), this average height will be used to evaluate the forces in the pylon under the various load cases.

2.3 Loading cases

With the symmetry of the deck on each side of the pylon, and with an adjusted tension of the cables, the pylon and the deck receive only normal force under permanent loads and connection type or inertia of elements don’t have any impact on this result. Some variation can appear with the longitudinal prestressing in the deck, but this case will not be studied here as too particular.

It is not the same under live loads as loading one span will produce forces and displacements in the deck and in the pylons. In order to compare the behavior of the bridge under live loads, the following characteristic loading cases will be studied on each model:
- A temperature variation of +30°.
- An uniform loading case corresponding to the sum of the lane load and pedestrian load of the Vietnamese regulation (total load 52.9 kN/m) with alternate spans loaded (case 1).
- The same uniform load with only half-span loaded as shown on fig. 2 (case 2).

3 PARAMETRIC STUDY

3.1 Studied parameters

The following parameters have been studied:
- • The foundation conditions with 2 cases: tower fixed in rotation at the bottom and foundation with rotation stiffness of 800000 MNm/rad.
- • The connection between the deck and the pylon with the 4 cases shown on fig. 3: a) deck simply supported on the pylon crossbeam by a rubber bearings line; b) deck embedded in the pylon (2 bearing lines and fixed longitudinally); c) tower embedded in the deck, fixed in rotation and free in translation on pier head; d) deck totally suspended with addition of cables.
- • The rigidity of the deck.
- • The rigidity of the pylon.

The results of this study are summarized in the following paragraphs.
3.2 *Analysis of the pylons under live loads*

3.2.1 *General behavior*

The forces in the pylon under live loads are coming from the stay-cables and from the deck.

If we suppose the deck flexible enough to neglect its bending moment, the sum of horizontal forces applied by the stay-cables is:

\[
H = \frac{qL^2}{8h}
\]  

where \(H\) is the horizontal force at the tower head, \(q\) is the uniform vertical load applied on the deck (52.9 kN/m), \(L = 300\) m and \(h = 61\) m.

For the load case 1, the value of \(H\) is 9.75 MN. This load, applied as a shear force to the tower, remains constant till the deck level. The values of forces on the pier are depending on the connection condition between deck and pylon:

- For the simply supported or totally suspended alternatives (cases a & d), no horizontal load is transmitted by the deck and the shear force remains constant till the foundation level.
- For the tower embedded in the deck with sliding support on the pier (c), the shear force is fully compensated by the normal force of the deck and the shear force disappears in the pier.
- For the deck fixed longitudinally and embedded in pylon (b), the deck transfers a part of the horizontal load to the other pylons and the shear force in the pier is inverted.

The bending moment curves result directly from these variations of the shear force and, curves’ shapes for central pylon are similar to those of the fig. 4 hereafter. Effects of parameters on the forces values will be analyzed in the following paragraphs.

3.2.2 *Effect of the foundations rigidity*

As expected, shear force and bending moment in pylons are reduced with more soft foundations. This reduction remains limited with only 7% for connection types a & d, 3% for c and about 0% for b.

The effect on horizontal displacement is more important, the displacement of the head of the tower being greater of 80 mm (+30%) for connection types a & d, 60 mm (+26%) for c and about 7 mm (+10%) for b.

3.2.3 *Effect of the deck rigidity*

The shear force, bending moment and displacement of the top of the tower are reduced with a more rigid deck. The table 1 presents the results of the shear force and displacement on the top of the tower with variable inertia referred to the original inertia \(I_0\).

3.2.4 *Effect of the pylons rigidity*

Shear force and bending moment in pylon are increased, but with reduced displacement of the tower head, when one increases the rigidity of the pylon. The table 2 presents the results of the shear force and displacement at tower head with variable inertia referred to the original inertia of the pylon \(J_0\).

3.2.5 *Effect of the connection type*

A comparison of the effect on pylons of the connection type with live loads has been carried out for the original structure. The main conclusions are as follows:

- Bending moment and shear force in tower are nearly the same under case 1 (difference less than 10%) and much smaller under case 2, see fig. 4,
Table 1. Horizontal force and displacement at central tower head depending upon deck inertia.

<table>
<thead>
<tr>
<th>H (MN)</th>
<th>0.5 I₀</th>
<th>I₀</th>
<th>2 I₀</th>
<th>4 I₀</th>
<th>8 I₀</th>
</tr>
</thead>
<tbody>
<tr>
<td>Connection a</td>
<td>8.03</td>
<td>7.67</td>
<td>7.19</td>
<td>6.52</td>
<td>5.63</td>
</tr>
<tr>
<td>Connection b</td>
<td>8.91</td>
<td>8.61</td>
<td>8.20</td>
<td>7.61</td>
<td>6.78</td>
</tr>
<tr>
<td>Connection c</td>
<td>8.19</td>
<td>7.67</td>
<td>7.26</td>
<td>6.62</td>
<td>5.81</td>
</tr>
<tr>
<td>Connection d</td>
<td>8.03</td>
<td>7.68</td>
<td>7.19</td>
<td>6.53</td>
<td>5.64</td>
</tr>
</tbody>
</table>

Disp. (mm) 0.5 I₀ | I₀ | 2 I₀ | 4 I₀ | 8 I₀ |
<table>
<thead>
<tr>
<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>Connection a</td>
<td>258</td>
<td>241</td>
<td>220</td>
<td>194</td>
</tr>
<tr>
<td>Connection b</td>
<td>211</td>
<td>193</td>
<td>175</td>
<td>156</td>
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<tr>
<td>Connection c</td>
<td>246</td>
<td>230</td>
<td>210</td>
<td>187</td>
</tr>
<tr>
<td>Connection d</td>
<td>258</td>
<td>241</td>
<td>220</td>
<td>195</td>
</tr>
</tbody>
</table>

Table 2. Horizontal force and displacement at central tower head depending upon pylons inertia.

<table>
<thead>
<tr>
<th>H (MN)</th>
<th>0.5 J₀</th>
<th>J₀</th>
<th>2 J₀</th>
<th>4 J₀</th>
<th>8 J₀</th>
</tr>
</thead>
<tbody>
<tr>
<td>Connection a</td>
<td>6.73</td>
<td>7.67</td>
<td>8.24</td>
<td>8.40</td>
<td>8.40</td>
</tr>
<tr>
<td>Connection b</td>
<td>7.92</td>
<td>8.61</td>
<td>9.07</td>
<td>9.30</td>
<td>9.40</td>
</tr>
<tr>
<td>Connection c</td>
<td>6.75</td>
<td>7.67</td>
<td>8.47</td>
<td>8.81</td>
<td>8.96</td>
</tr>
<tr>
<td>Connection d</td>
<td>6.74</td>
<td>7.68</td>
<td>8.25</td>
<td>8.41</td>
<td>8.40</td>
</tr>
</tbody>
</table>

Disp. (mm) 0.5 J₀ | J₀ | 2 J₀ | 4 J₀ | 8 J₀ |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Connection a</td>
<td>374</td>
<td>241</td>
<td>134</td>
<td>69</td>
</tr>
<tr>
<td>Connection b</td>
<td>295</td>
<td>193</td>
<td>114</td>
<td>65</td>
</tr>
<tr>
<td>Connection c</td>
<td>363</td>
<td>230</td>
<td>127</td>
<td>67</td>
</tr>
<tr>
<td>Connection d</td>
<td>374</td>
<td>241</td>
<td>134</td>
<td>69</td>
</tr>
</tbody>
</table>

- The difference in the pier corresponds to the behavior explained in paragraph 3.2.1 with a maximum bending moment at foundation level of 760 MN.m for connections a & d, 460 MN.m for connection c and 170 MN.m for connection b.
- The displacement of the tower head is similar for connection type a, c and d, and smaller for connection type b.

3.2.6 Conclusion on pylons under live loads

The parametric study shows that the formula (1) gives shear forces in excess of 10 to 25% of the forces in the pylons show small variations with the deck or pylon rigidity. Forces in pier and load applying on foundation are mainly dependent upon the connection type. Displacement at tower head is mainly dependent upon pylons rigidity.

3.3 Analysis of the deck under live loads

3.3.1 General behavior

We will analyze in the following paragraphs the effect of the variable parameters on the deck design. We will look in particular the maximum and minimum bending moment, the deflection and the shear force at mid-span.

One of the particularities of the multiple cable-stayed spans bridges is their large shear force at mid-span under the live load case 2, as this force shall balance partially the unbalanced loads. If we consider that all unbalanced loads are taken by shear force at mid-span, its value can be calculated according to the following formula:

\[
V = \frac{qL}{8}
\]

with use of the previous notations. For the load case 2, this value of V is 1.98 MN.

3.3.2 Effect of the foundations rigidity

The effect of the foundations rigidity on horizontal displacement of the tower head has been analyzed previously and will have direct effect on the deck forces. The table 3 summarizes the main results for the main spans of the deck.

The reduction of the foundation rigidity induces in the deck greater deflections (about same ratio than for the horizontal displacement of the tower head) and greater bending moments (between +10 and +20%), except for the case b: deck embedded on the tower.

For this case, we can understand that the deck working together with pylons transfers unbalanced loads to the adjacent pylon, and that system avoids additional flexure.

3.3.3 Effect of the deck rigidity

As expected, the bending moments in the deck increase with the rigidity of the deck, as shown on the table 4 hereafter.
Table 4. Bending moment of the deck depending upon deck inertia.

<table>
<thead>
<tr>
<th>Connection</th>
<th>0.5 I₀</th>
<th>I₀</th>
<th>2 I₀</th>
<th>4 I₀</th>
<th>8 I₀</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>30</td>
<td>36</td>
<td>50</td>
<td>75</td>
<td>110</td>
</tr>
<tr>
<td>b</td>
<td>20</td>
<td>27</td>
<td>40</td>
<td>62</td>
<td>94</td>
</tr>
<tr>
<td>c</td>
<td>20</td>
<td>28</td>
<td>44</td>
<td>66</td>
<td>98</td>
</tr>
<tr>
<td>d</td>
<td>30</td>
<td>36</td>
<td>50</td>
<td>75</td>
<td>108</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Connection</th>
<th>0.5 I₀</th>
<th>I₀</th>
<th>2 I₀</th>
<th>4 I₀</th>
<th>8 I₀</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>−27</td>
<td>−32</td>
<td>−39</td>
<td>−55</td>
<td>−76</td>
</tr>
<tr>
<td>b</td>
<td>−17</td>
<td>−22</td>
<td>−30</td>
<td>−43</td>
<td>−75</td>
</tr>
<tr>
<td>c</td>
<td>−22</td>
<td>−23</td>
<td>−35</td>
<td>−46</td>
<td>−76</td>
</tr>
<tr>
<td>d</td>
<td>−28</td>
<td>−33</td>
<td>−49</td>
<td>−56</td>
<td>−80</td>
</tr>
</tbody>
</table>

Note: Maximum bending moment for 0.5 I₀ and I₀ are given for all connection types by live load case 2.

Table 5. Bending moment of the deck depending upon pylon inertia.

<table>
<thead>
<tr>
<th>Connection</th>
<th>0.5 J₀</th>
<th>J₀</th>
<th>2 J₀</th>
<th>4 J₀</th>
<th>8 J₀</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>47</td>
<td>36</td>
<td>35</td>
<td>35</td>
<td>35</td>
</tr>
<tr>
<td>b</td>
<td>37</td>
<td>27</td>
<td>22</td>
<td>18</td>
<td>16</td>
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<tr>
<td>c</td>
<td>40</td>
<td>28</td>
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<td>18</td>
<td>15</td>
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<tr>
<td>d</td>
<td>47</td>
<td>36</td>
<td>35</td>
<td>35</td>
<td>35</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Connection</th>
<th>0.5 J₀</th>
<th>J₀</th>
<th>2 J₀</th>
<th>4 J₀</th>
<th>8 J₀</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>−43</td>
<td>−33</td>
<td>−33</td>
<td>−32</td>
<td>−31</td>
</tr>
<tr>
<td>b</td>
<td>−33</td>
<td>−22</td>
<td>−18</td>
<td>−18</td>
<td>−18</td>
</tr>
<tr>
<td>c</td>
<td>−36</td>
<td>−23</td>
<td>−20</td>
<td>−20</td>
<td>−20</td>
</tr>
<tr>
<td>d</td>
<td>−43</td>
<td>−33</td>
<td>−33</td>
<td>−32</td>
<td>−31</td>
</tr>
</tbody>
</table>

Note: Bending moment given for all connection types by live load case 2.

Figure 5. Bending moment in the deck (half symmetrical part) under case 1 (top figure) and case 2 (bottom figure).
For small inertia of the deck, the maximum forces are given by the live load case 2 (half spans loaded). This type of case has to be considered in the design of such bridge.

With the increase of the deck rigidity, the shear force at mid-span does not change for connection type a and d, and is variable from 1.4 MN till 1.9 MN for b and c. The deflection at mid-span is variable from 330 till 580 mm for connection type a and d, from 270 till 460 mm for b, and from 280 till 480 mm for c.

3.3.4 Effect of the pylons rigidity
The table 5 presents the variation of the maximum and minimum bending moment in the deck when the rigidity of the pylon varies.

We would expect a decrease of the bending moment in the deck with the increase of the pylons rigidity, but due to the live load case 2, the bending moment remain constant for connection type a and d.

With the increase of the pylons rigidity, the shear force at mid-span does not change for connection type a and d, and is variable from 1.7 MN till 0.9 MN for b and c. The deflection at mid-span is variable from 760 till 180 mm for connection type a and d, from 580 till 170 mm for b, and from 620 till 170 mm for c.

The increase of the rigidity of the pylons has an important consequence for the deflection of the deck, but only a small effect on the forces.

3.3.5 Effect of the connection type
From the tables above and fig. 5 hereunder, we can understand the effect of the connection type on the deck. Connection a and d are similar for the forces and displacement.

Connection type b and c allows a decrease of forces of about 20~30% in the deck under live loads.

3.3.6 Conclusion on deck under live loads
We can decrease the forces in the deck under live loads by choosing the connection type (type c is preferred) and by reducing the deck rigidity. Displacement can be reduced by connection type (c preferred) and by increasing the tower rigidity.

For standard or low rigidity of the deck, the designing case would be the live load case 2 (half spans loaded).

3.4 Analysis of the behavior under thermal effect
3.4.1 Analysis of the deck
The table 6 summarizes the main forces in the deck under a thermal load of +30°C.

The connection types a and d behave in the same way with a normal force and negative bending moment increasing with the rigidity of the tower.

The connection type b gives a heavy normal force in the deck in all cases and if we want to apply this system we shall allow the possibility of displacement under long time variations, such as thermal variation, to keep an economic alternative.

The better way is to consider a connection type c which gives smaller bending moments and normal forces.

3.4.2 Analysis of the side pylon
The table 7 summarizes the main forces in the side pylon under a thermal load of +30°C.

The connection types a and d behave in the same way with a shear force increasing with the rigidity of the tower and a displacement decreasing.
The connection type b gives a heavy shear force in the pylon in all cases, that also required for allowing long time displacement of the deck.

The better way is also to consider a connection type c which gives smaller shear force and smaller displacement. For case c, the displacement at deck level between tower and pier, evaluated at 215 mm, has not been added to the displacement of tower head.

4 CONCLUSION

The main conclusions of this study about behavior of multiple cable-stayed spans bridges under live loads and thermal variation are:

- The connection type c (tower and deck sliding on pier) is the more effective and economic for the studied load cases.
- For the connection type c, it is efficient to reduce the deck rigidity and to increase the pylon rigidity.
- The connection type b (deck embedded in the pylon) can be more efficient, but we shall solve the problem of the extension under long time variations.
- The type a and d structures (deck simply supported on pylon or fully suspended) are less efficient under live loads and thermal variation, with more forces on the foundations and more force in the deck.
- Live loads case with half spans loaded (case 2) shall be studied for the design as it appears to be designing case at deck mid-span.

In this study, we analyzed live loads and thermal variations only, but other effects like seismic effect shall be studied.

REFERENCE
