The Design of Steel-Concrete Hybrid Portal-frame
In Beijing-Shanghai High-speed Railway, China

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Abstract: This paper describes a design of steel-concrete composite portal-frame in Beijing-Shanghai High-speed Railway which is located at Shanghai Hongqiao. With the comparison of pre-stressed concrete as well as steel structure, composite structure has been chosen. In this way, the structure can keep the lower weight as well as good appearance by inserting the slight expansive concrete in composite section. Meanwhile, difficult construction condition could be avoided by prefabricating and hoisting the steel superstructure. During the design process, the code in England (BS5400), Japan, USA, German, as well as China have been used. Since the portal-frame adopted steel pier-beam consolidated system while a part of steel column inserted into concrete pier, the difference between these codes led the problem such in the disposal of the steel-concrete joint part, the arrangement of stiffening rib and the layout of shear studs. Through this design, the difference of steel structures Codes between China, Japan, German, USA as well as Britain has been studied.
1. Introduction

Beijing-Shanghai High-speed Railway with the speed of 350km/h covers about 1318km, including 244 bridges with a total length of 1060km. This new line need to cross a number of existing railways. Portal-frame piers are widely applied in high-speed railway bridges spanning the existing railways, which can step over the existing lines flexibly, and provide more space for driving of existing lines. Especially, steel and steel-concrete hybrid portal-frames could adopt the method of factory pre-fabrication and on-site assembly, with less restrictive conditions and shorter construction period. This paper is based on the 32-38# portal-frame piers of Yun Zao Bang Bridge near Hongqiao Station. The original design is pre-stressed concrete; afterwards, structure forms are modified considering restrictive construction conditions and tight schedule (Fig.1).

![Figure 1: Plan layout of the railways](image1)

At first, three kinds of structures has been chosen, including pre-stressed concrete piers, steel piers, and hybrid portal-frame. By comparison on the safety, cost, construction period and other aspects, steel-concrete hybrid has been chosen finally, with two types of spans 29.3m and 30.8m, using steel-box section with the height of 3.25 and width of 3.3m (Fig.2).

![Figure 2: Schematic drawing of 38# portal-frame pier](image2)

The main characteristics of the structure are as follows: All-steel coping, steel-concrete composite column, steel pier-beam consolidated system while concrete is poured outside and inside the steel column in the 4.5m steel column inserted into the concrete pier.
Meanwhile, shear studs are laid on the inner wall of steel columns with expansive concrete inside the steel columns. Bridges built on the portal-frame piers are 24-meter simply supported beams under the ZK load. Internal force of each pier varies due to different location where the loads act on, so the thickness of steel plate is selected according to the actual condition. During the design, existing steel structure design codes of several countries has been referred, and the selection of the stiffening ribs, the layout of shear studs, the disposal of steel-concrete joint part and calculation check of structural displacement are studied and discussed as follows.

2. Design of joint part

Hybrid portal-frame piers with the consolidation of steel capping and composite structure pier are adopted, so as to construct conveniently and reduce cost. What are key issues in design are how to ensure that the loads on steel girders could be transferred to composite piers effectively, and how to dispose the joint part of steel and concrete structures.

2.1 Selection of connection method

Girders and piers could be joined by steel bars, anchor bars and steel column. In this design, girders and piers are joined by the use of steel columns. Firstly, steel columns and girders are made as a whole. Then, the steel columns are inserted into reinforced concrete piers directly. The steel columns are combined with the concrete piers through cylinder head studs. Generally, the length of inserting part of the steel column is 1.5 times of its short side. Inside the steel columns, concrete is casted to make the structure more stable. The load transmission routes are in subsequence of steel girders, steel columns, shear studs, concrete, steel bars and concrete piers. Advantages are as follows: the steel columns can be used as pile caps to adjust the main girders elevations of each span, meanwhile, the construction becomes more convenient as there are no steel bars extending into the joint part.

2.2 Design and Arrangement of shear connectors

To ensure a full composite action, shear connectors should be provided at the interface between the concrete and the steel to resist interface shear. In this design, cheese head studs were adopted as the shear connectors of the steel structure and concrete pier. According to the actual internal force of the pier, shear studs are set up in the 4.5m steel pier which is inserted into concrete pier. Meanwhile, studs are laid in lower margin of the bottom plate, which is located at the intersection of the steel beam and steel pier. Shear studs were arranged as the regulation of Japanese codes, and checked according to Chinese Code for design of steel structures (GB50017-2003) and American AASHTO LRFD.

2.2.1 Calculation method on studs in Japanese codes

Japanese code uses allowable stress method to calculate shear resistance of studs, calculation formulas should be selected according to the ratio of height and diameter of the stud: $Q_s = 30d^2 \sqrt{\frac{\sigma_{as}}{d}} (H/d \geq 5.5)$, or $Q_s = 5.5d \cdot H \sqrt{\frac{\sigma_{as}}{d}} (H/d < 5.5)$ (1)

where: d is the diameter of the stud, H is the height of the stud.
The arrangement of studs of this structure is carried out in accordance with the above formula: \( H / d = \frac{12.5}{2.5} = 5 \leq 5.5 \), so \( Q_a = 5.5 \cdot 2.5 \cdot 12.5 \cdot \sqrt{340} = 31.69 \text{KN} \). After the internal forces of the left and right steel piers are calculated, the required minimum number of shear studs can be known. Following the constructional requirement of codes, then we can arrange the shear studs. Considering the force characteristics of portal frame structure, studs should be laid densely on the top and sparsely on the lower part in the height direction, meanwhile studs of the left and right web plates ought to be set more densely than that of the front and rear web plates. Layout of studs can be seen in Fig.3.

![Figure 3: Schematic drawing of the layout of studs in the steel pier](image)

### 2.2.2 Checking method about studs in Chinese codes

A provision is provided in Code for design of steel structures (GB 50017/2003): Shear connectors of composite structure should select studs, but channel steel, bent bar, and other kind connectors with reliable basis are also can be used. The ultimate bearing capacity of single cheese head stud should be calculated as the following formula:

\[
N_v^c = \frac{0.43 A_s E_c f_c}{A_c} \leq 0.7 \gamma f
\]

where:
- \( E_c \) is the modulus of elasticity of concrete
- \( A_s \) is area of component of steel section
- \( \gamma \) is the ratio of the minimum tension strength and yield strength of the stud material
- \( f \) is the design value of the tension strength of the cheese head stud.

The concrete used in these portal frame piers is concrete C40, the steel plates are all made from Q345qD, the strength of single stud can be calculated by above formula. Therefore:

\[
N_v^c = \frac{0.43 A_s E_c f_c}{A_c} = \frac{0.43 \cdot 0.25 \cdot \pi \cdot 25^2 \cdot \sqrt{27 \cdot 34000}}{\pi} = 202.24 \text{KN}
\]

### 2.2.3 Checking method about studs in American codes

Horizontal shear is demanded to be calculated and checked according to the AASHTO2005, which adopts ultimate strength method to calculate the number of studs, based on the ultimate state when the steel plate or concrete was damaged. The strength of one stud

\[
N_v^c = 0.5 \cdot A_s \sqrt{E_c \cdot f_c} = 235.2 \text{KN}
\]

number of required studs \( n = V_h / 0.85 N_v^c \).
\[ V_h = \text{smaller} \left\{ 0.85 f'_u b_{eff} t_s \left( \sum A_{si} F_{yi} \right) \right\}. \]  

### 2.2.4 Analysis and Comparison

From the comparison mentioned above, it is indicated that the theoretical basis of them are different. Based on linear elasticity theory, allowable stress method was used in Japanese code, which adopts the criterion that calculated stress of certain point or part on the structure’s dangerous section should be no more than allowable stress of the material. This method is simple and clear, but it is relatively conservative and it cannot provide a consistent safety factor. Unlike Japanese code, Chinese and American code adopt ultimate strength method to calculate the stud’s ultimate bearing capacity, under the failure state of the material, with a certain safety factor considered.

<table>
<thead>
<tr>
<th>item</th>
<th>code</th>
<th>Japanese codes</th>
<th>Chinese codes</th>
<th>American codes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Theory base</td>
<td></td>
<td>allowable stress method</td>
<td>ultimate strength method</td>
<td>ultimate strength method</td>
</tr>
<tr>
<td>Basic requirement</td>
<td>longitudinal spacing≥6d, transverse spacing≥4d</td>
<td></td>
<td>longitudinal spacing≥4.5d, transverse spacing≥4d</td>
<td>h/d≥4, transverse spacing≥4d</td>
</tr>
<tr>
<td>Formula</td>
<td>( Q_a = 30d^2 \sqrt{\sigma_{ck}} )</td>
<td>( Q_a = 0.43 \cdot A_s \sqrt{E_c \cdot f_c} )</td>
<td>( 0.5 \cdot A_s \sqrt{E_c \cdot f_c} )</td>
<td></td>
</tr>
<tr>
<td>Shear resistance</td>
<td>31.69kN</td>
<td>202.24 kN</td>
<td>235.2 kN</td>
<td></td>
</tr>
</tbody>
</table>

Table 1: Comparisons of clause on studs between several countrys’ codes

As shown in Table1, the formulas that calculate the ultimate bearing capacity of studs in Chinese codes and American codes are very similar, and calculate results are very close, while the calculated allowable stress by Japanese code is only about 1/6 of them. That is to say, the safety factor of Japanese code is six times higher than that of ultimate strength method, as it has taken the uncertainty of the strength of studs into consideration. If the shear studs could meet the requirements of Japanese codes, fatigue strength of studs needs no checking. However, experiments show that the actual bearing capacity of studs is much greater than the result obtained by elasticity method. Consequently, the rational calculation of shear studs should select the ultimate strength method in practical project.

### 3. Design of steel beams

How to select and lay out stiffening ribs of the section is the focus of structure design, which is related to the stability and safety of the structure directly. The stiffening ribs of the section must satisfy the requirements of local stability in order to avoid structural local buckling. In this design, the steel box section with large inertia moment and better mechanical performance is selected, and the section stiffener is taken in the form of flat stiffener with thickness of 18mm and length of 180mm. The stiffening ribs are laid out
symmetrically, and set completely the same between the top and bottom floor, which can make the same section capacity of resisting positive and negative bending moment.

3.1 Requirements of stiffening rib settings in Japanese codes

The stiffening ribs required in the Japanese codes must meet the following conditions:

1. The thickness of single or bilateral stiffened compression plates must be greater than the specified value (the limit is related with the type of steel). However, the stiffened plates which bear compressive stress temporarily in the process of erecting only need to meet $t \geq \frac{b}{80} f n$, where, $f = 0.65 \left( \frac{\sigma}{n} \right)^2 + 0.13 \left( \frac{\sigma}{n} \right) + 1.0$, $\varphi = \frac{\sigma_n - \sigma_t}{\sigma}$, $b$ is the width of stiffening plates, $f$ is stress gradient, $n$ is the number of plate grid, and the space of stiffening ribs can be assumed to be equal and evenly distributed (Fig. 4).

2. The steel of longitudinal stiffening ribs must have the same or larger level with the steel stiffened.

3. The second moment of stiffening ribs must be calculated according to the following regulations: When the stiffening ribs are set at one side of the plate stiffened, the second moment is the calculated value for the surface of the plate stiffened with the stiffening ribs; When the stiffening ribs are set at both sides of the plate stiffened, the second moment is the calculated value for the center of the plate stiffened.

4. According to (3), the second moment $I_i (cm^4)$ and area $A_i (cm^2)$ must meet the following requirements: $I_i \geq \frac{bt^3}{11} \gamma_{1, reg}$, $A_i \geq \frac{bt}{10n}$.

Where:
- $t$ is thickness of stiffening plates (cm)
- $b$ is width of stiffening plates (cm)
- $n$ is number of plate grid divided by longitudinal stiffening ribs
- $\gamma_{1, reg}$ is stiffness ratio of longitudinal stiffening ribs calculated according to the formula, which is related to the spacing of the stiffening rib.

3.2 Requirement of stiffening rib settings in British codes

The stiffening ribs required in British codes BS5400 must meet the following conditions:

1. $h_i \sqrt{\frac{\sigma_{yx}}{355}} \leq 10$, (2) $h_i \sqrt{\frac{\sigma_{yx}}{355}} \leq 31$, $h_i \sqrt{\frac{\sigma_{yx}}{355}}$ cannot be more than the...
corresponding limit (according to the chart), as shown in the BS5400 (1982) 9.3.4.1.2. where:

B is spacing of stiffeners, or the distance between stiffeners and beam flanges

t is thickness of plats, \( h \) is height of stiffening plates

\( t_s \) is thickness of stiffening ribs, \( \sigma_{ss} \) is nominal yield stress of stiffening plates

\( \sigma_y \) is nominal yield stress of flanges, webs or hollow sections.

3.3 Comparison

The requirements of stiffening ribs in British codes focus on slenderness ratio of members, which is considered referred to members’ local buckling; while, those in Japanese codes emphasize on the restrictions of inertia moment and area of the section. Comparatively speaking, the requirements in Japanese codes are much stricter, as it not only set strict rules on stiffeners, but also put forward a series of requirements to stiffened steel plates. Thus, a higher safety factor is achieved, the Japanese codes is more conservative. However, the checking requirements of the two codes on the stiffening ribs are not considered to be comprehensive, only focused on the unilateral, and to a certain extent, there is even conflicting aspects. For example, the longer the member bar, the greater the inertia moment and the area, the more likely to meet Japanese standards. But when the slenderness ratio is too large, it is unlikely to meet the British standards, and does not conform to the classical theory and experience. On the contrary, the inertia moment and section area would be difficult to meet requirements of Japanese codes, if slenderness ratio is strictly limited.

The codes in future should establish the new unified standard, based on the coordination of the two above. As far as this design, the sizes of the stiffening ribs are preliminarily determined according to British specification, and then, adjustments are gradually carried out to meet the Japanese codes. In this way, the slenderness ratio of stiffening ribs is less than the limit value, meanwhile, the inertia moment and area can also achieve certain requirements. Thus, the local stability of the structure can be fully guaranteed. Through analysis and comparison, it is showed that U/type stiffener tend to meet the two nations’ codes more easily, with better local stability than flat stiffener and T-type stiffener.

4. Deformation limit of portal-frame piers

4.1 Control standard of pier deformation in Chinese codes

Deformation of piers is required to meet the following regulations in Beijing-Shanghai high-speed Railway Tentative Provision:

(1) Requirement of track regularity under live load: For a 3m-long railway, relative vertical deformation of two rails of one line should not be greater than 1.5mm under the ZK live load, and not be greater than 1.2mm under the action of actual trains (Fig.5).

(2) Considering the effect of ZK live load, temperature load, lateral swing force, centrifugal force, and wind force, the horizontal angle at the beam end of the deck, which is caused by horizontal elastic displacement of pier top, should be no greater than 1‰.

(3) The settlement of the foundation of piers ought to be calculated under the dead load, the settlement after construction should meet the limit value in Table2.
<table>
<thead>
<tr>
<th>item</th>
<th>ballasted deck bridge</th>
<th>ballastless deck bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniform settlement of piers</td>
<td>≤30mm</td>
<td>≤20mm</td>
</tr>
<tr>
<td>Difference between adjacent piers</td>
<td>≤15mm</td>
<td>≤5mm</td>
</tr>
</tbody>
</table>

Table 2: The limit of settlement of piers

4.2 Control standard of pier deformation in German codes

(1) In the railway section whose design speed is greater than 160km/h, level angle of adjacent spans must not exceed 1‰, load ought to be considered include centrifugal force, lateral impact force, wind force of piers, beams body and vehicle on the bridge, temperature difference of piers and beams, as well as rotation caused by foundation displacement.

(2) Under the effect of UIC load, the ratio of deflection to span in the transverse direction should be less than or equal to 1/4000.

(3) Using UIC71 static load and impact factor to calculate the deformation of the track, the maximum deformation of a 3m-long railway is required to be less than the limit value in table 3. If the train speed is greater than 220 km/h, dynamic analysis should be carried out in order to ensure the maximum deformation of the track is less than 1.2 mm/3m.

4.3 similarities and differences

Ensuring passenger travelling comfort and track regularity is important principle during the design of high-speed railway bridges. By comparing specifications between China and Germany, checking contents of the deformation of portal/frame piers are extremely similar, track regularity and pier’s transverse rigidity are restricted and the limit values of the two codes are very close. From table 6, it can be concluded that the two codes are unified, but as ZK load is equivalent to 0.8 UIC load, in some sense, the deformation restrictions of German code is stricter than that of Chinese code.
Table 4: Comparison of provisions on track regularity of Chinese code and German code

<table>
<thead>
<tr>
<th>item</th>
<th>Chinese codes</th>
<th>German codes</th>
<th>relative value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kind of live load</td>
<td>Zk live load</td>
<td>UIC live load</td>
<td></td>
</tr>
<tr>
<td>concentrated force of load</td>
<td>200</td>
<td>250</td>
<td>0.8</td>
</tr>
<tr>
<td>uniform force of live load</td>
<td>64</td>
<td>80</td>
<td>0.8</td>
</tr>
<tr>
<td>track deformation- t</td>
<td>≤1.5mm</td>
<td>≤1.5mm</td>
<td>1</td>
</tr>
</tbody>
</table>

5. Conclusion

Currently, design codes of steel structures vary from nation to nation. The bridge constructions of China started late, thus, inexperience and roughness still exist in the specifications. Besides, many important local structures are not involved in the specifications. So, improvements and perfections are still needed. The U.S. specification, which adopts ultimate strength method, is suitable for calculations in practical project, because of its clear formulas and consistence with experimental results. While, the Japanese specification, using allowable stress method, most of which are summarized and refined from the actual projects, biased towards safe and conservative, can be considered as a check-calculation criteria of structural safety.

Appearance design of bridges is developing towards the diversification, but the standardization of structures and unionization of design codes will also be inevitable result of the development of bridge design. Several different factors such as geomorphic condition, traffic demand, and construction level lead the differences between the design codes of different countries. However, the safety of the structure, which is the most important principle of bridge design, decides that codes of different countries shall be interoperable and harmonious in principle. Science and technology have no national boundaries, future design code should be the unity of different codes by comparing with others and widely learning advantage from others, so that it can be used widely in actual project, and meanwhile ensure the structure is more safe, economical, and durable.

REFERENCES