Design of A Special Bridge for Taiwan High Speed Rail Project Lot C296

Szu-Ming Kang
Structural Engineer
Moh And Associates Inc.
Taiwan, R.O.C.

S.M Kang, born 1969, received his Civil Engineering degree and M.Sc. degree both from National Cheng Kung University, Taiwan. His main experience is in the design of bridge engineering in highway, MRT and high speed rail project.

Hsiu-Kang Peng
Project Engineer
Moh And Associates Inc.
Taiwan, R.O.C.

H.K Peng, born 1963, received his Civil Engineering degree from Feng Chia University, Taiwan. His major experience is in the steel structure design for bridges and building structures. He is the project Engineer for HSR C296 Contract.

Huat-Yoo Chua
Manager,
Moh And Associates Inc.
Taiwan, R.O.C.

H.Y. Chua, born 1955, received his B.S.C.E. from National Taiwan University, Taiwan and M.Eng (Structural) degree from AIT, Thailand. As a Chartered Engieer (U.K)., he has devoted all his time in the planning ,design, and contract administration .in high-rise buildings, bridges and underground structures.

Summary

Because of the lightweight nature of steel structure, the steel bridge of DU-108 is adopted to cross the No. 1 Provincial Highway for 90 m long span, and it performs well as the concrete bridges for all the functional requirements of HSR. Various aspects in analysis and design are carried out for the special bridges, such as dynamic analysis of rolling stock, track-structure interaction, fatigue stress check etc., besides the anti-seismic consideration and stringent functional requirements of the HSR.

Keywords: Taiwan High Speed Rail (THSR); special bridges; steel box girders, steel piers, service load design method, fatigue, track-structure interaction, passage of rolling stock

1. Introduction

The Taiwan High Speed Rail (HSR) runs approximately 345km from Taipei at the north to Kaohsiung at the south of the Formosa Island. Currently under construction, this BOT project is believed to be the largest of its kind in the region. The civil works of the Taiwan HSR project have been divided into twelve separate contracts, whereas Lot C296 is the last section at the southern end (see Fig. 1). This section of HSR passes through Zen-Wu Village of Kaohsiung County and Tsoyin area of Kaohsiung City. The total length is about 3 km with about 2km of alignment lying in-line with Water Pipe Road, of which massive utility pipeline running underneath. The structural systems in this section are viaducts comprising about 2.3 km long prestressed concrete box girder bridges and special bridges, including three 200 m long steel box girder bridges. The three steel box girder bridges with 55m+90m+55m spans across the Zhong-Shan freeway and the two main roads in that area, Fengjen Road (County Road 183) and No. 1 Provincial Highway, respectively.

Fig. 1 Location of the HSR C296 Contract

These special bridges are adopted not only due to the structural functional requirements but also the construction considerations. Various aspects in analysis and design are carried out for the special bridges, such as dynamic analysis of rolling stock, track-structure interaction, fatigue stress check etc., besides the anti-seismic consideration and stringent functional requirements of the HSR.

This paper describes in full detail with special discussion on major issues encountered during the design works for one of the special bridges, Design Unit 108 (DU-108), crossing the No.1 Provincial Highway.

2. Bridge description and structural system

Bridge DU-108 spans across the No. 1 Provincial Highway of 60m wide. While on a horizontal curve of 600 m in radius, the bridge layout is a three spans continuous steel box girders of 55+ 90 +50 m with two intermediate steel piers (P71, P72) and two side concrete piers (P70, P74). The steel box girders are integrated with the steel pier caps as a rigid connection. Both end of this bridge are connected to 25 m simply supported concrete box girder bridges. The structure layout is shown in Fig. 2. The bridge structural system is described as follows: Superstructures of three steel box girders of 2.3 m wide with depth varying from 5.0 m at intermediate rigid support connections to 4.0 m at mid-span and side spans. The thickest steel plates adopted in flanges are 40 mm and in webs are 22 mm to ASTM A709 GR. 50. Reinforced concrete bridge deck of 27 cm thick with shear connectors to ASTM A108 are provided, to achieve the composite action between the concrete deck and steel girders. It is necessary and effective for sharing the horizontal forces and promoting the total rigidity to fulfil the strict requirements of deflection and rotation limits for HSR structures. Fig. 3 shows the typical section of the superstructure and Fig.4 shows the typical section of the piers.

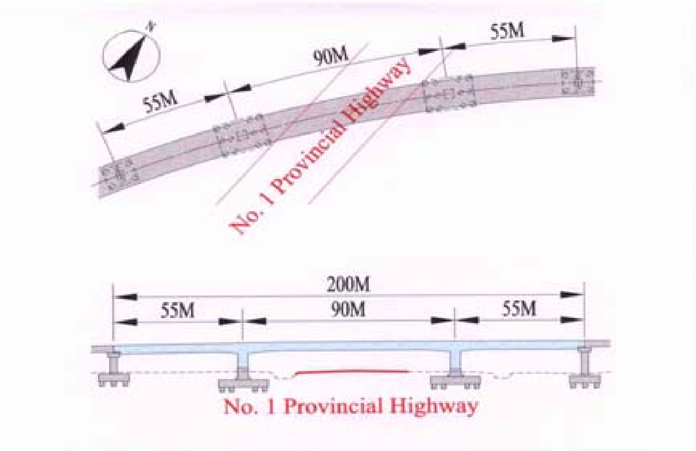


Fig. 2 Structure layout

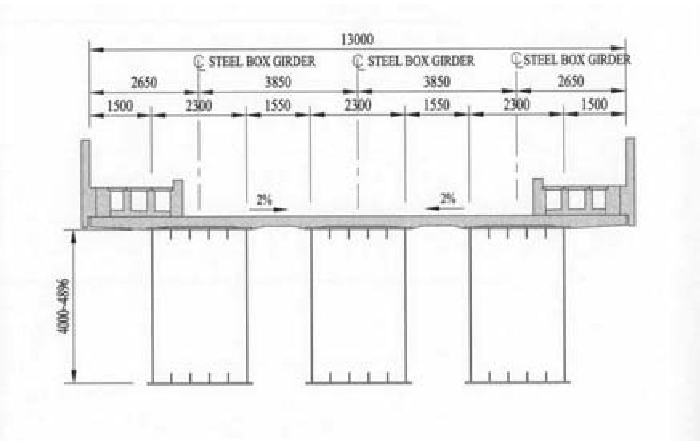


Fig.3 Typical section of superstructure

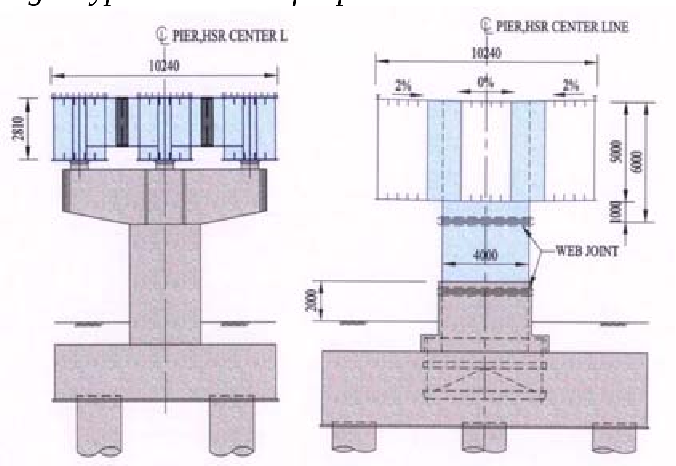


Fig. 4 Typical section of interior and exterior piers

Interior piers P71, P72 of steel box section are 4.0 m x 4.2 m wide with internal stiffener being provided. The thickest steel plates adopted in flanges and webs are 60 mm. The connections between steel girders and steel pier caps are rigid connections.

Side piers P70 and P74 of reinforced concrete are 3.4 m x 3.0 m wide. The connections between both ends of exterior supporting piers and superstructures are provided by guided pot bearings moveable in longitudinal direction and constrained in transversal direction

The pier structures are supported by bored pile foundation.

The main reasons of the steel bridge structural system mentioned above being selected for DU-108 are listed as follows:

-Because of the lightweight nature of steel structure and integrating the low vertical profile limitation and clearance

-Requirement of the existing road underneath the HSR line can be achieved for 90 m span.

-Steel box girders have good torsional rigidity performance for curved bridge alignment.

-The traffic and environmental impact at the wide road junction can be reduced effectively because of the fast erection characteristic of steel structures.

-From the aesthetic point of view, the steel bridge looks less heavy than concrete bridge.

3. Longitudinal structural analysis and design

3.1 Design Standards and Codes

The major design specification, standards and codes for analysis and design works includes THSRC Design Specifications issued, January, 2001 [1], local Seismic Design Specification for Railway Bridges, and the Design Specification for Highway Bridges, 1982 [2,3], etc. In addition, those AASHTO [4] and Japanese codes [5] are also referred to supplement the local design codes.

3.2 Materials.

Steel Plate: ASTM A709 Grade 50, Fy ≧ 350 MPa

High Strength Bolts: ASTM A490-Type 3

Shear Studs: ASTM A108

Anchor Bolt: JIS S35C(N)

Concrete:

fc'= 35 MPa for bridge deck

fc'= 28 MPa for concrete piers and foundations

3.3 Structural analysis model and design loads considerations

A 3-D frame analysis model is generated by SAP 2000 program, which consists of the elements of box-girders, transverse frames, pier columns and foundation springs (see Fig. 5). The position of mass loading applied to structural model is in accordance with the actual condition for more accurate results. For example, the mass centre of gravity of one train loading is applied at the location of 2.56m vertically upward and the horizontal eccentricity with 2.25m to the centre of bridge deck. There are altogether eleven spans (three main spans and eight adjacent spans) modelled in order to prevent underestimate of earthquake forces in transverse direction. The design loads are considered in accordance with the Design Specifications of THSRC, including dead load, superimposed dead load, train live load, centrifugal force, braking and acceleration force, wind load, temperature effect, seismic load, etc. Seismic design considerations are particularly critical for HSR structures in Taiwan, and it is based on an earthquake with a return period of 950 years.

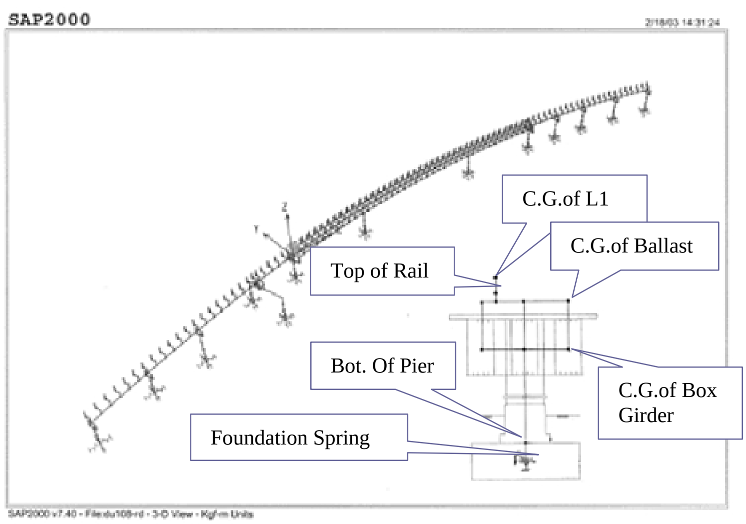


Fig. 5 Structural Model

The dynamic analysis method is

employed for the aforementioned seismic loading condition in accordance with the Design Specifications of THSRC and the local Seismic Design Specification for Railway Bridges. More specifically, the multi-model response spectrum

analysis has been employed for the dynamic analysis.

The natural periods of the first modes in transverse, vertical and longitudinal directions are 0.81 second, 0.72 second and 0.56 second respectively.

3.4 Structural Design of The Steel Bridge

Service load design (Working Stress Design) method is adopted for the design of the steel bridges in C296. All structural functional requirements including strength demands and deflection limitations specified by aforementioned specifications are fulfilled for all structural components of the steel bridge, they are described item by item in the following sections.

(1) Deflection limitations

The vertical and lateral angular deformations of the steel bridge by considering one train of modified UIC loading with impact factor has been well controlled to be in accordance with the limitations as followings:

The maximum deflection of the bridge should be less than 1/2755 of the span length.

The maximum lateral angular rotation of the bridge should be less than 0.0013 radians.

(2) Design of Steel Box Girders

The stress level for each component of the steel box girders has been checked to meet its allowable stress level, including the tensile and shear stresses of flanges and webs, slip resistance of high tension bolts of field splices, horizontal shear stress of shear studs, etc. Besides, the longitudinal, transverse and vertical stiffeners for flanges and webs of steel box girders are properly located to avoid local buckling of the steel plates.

(3) Design of Steel Piers

The steel piers are subjected to the axial compression forces, bending moments in two-direction and horizontal shear forces simultaneously so the combined effects by these forces components shall be considered. The combined stress requirements specified by AASHTO and aforementioned Japanese codes have been satisfied for the sections of interior steel pier. The longitudinal stiffeners for flanges and webs are also properly located to avoid local buckling of the steel plates. Furthermore, shear lag effect on steel pier section cannot be ignored and thereby has been properly considered in the design. The conjunction area of the steel column and cap need to be strengthened due to its complex stress distribution and shear lag effect.

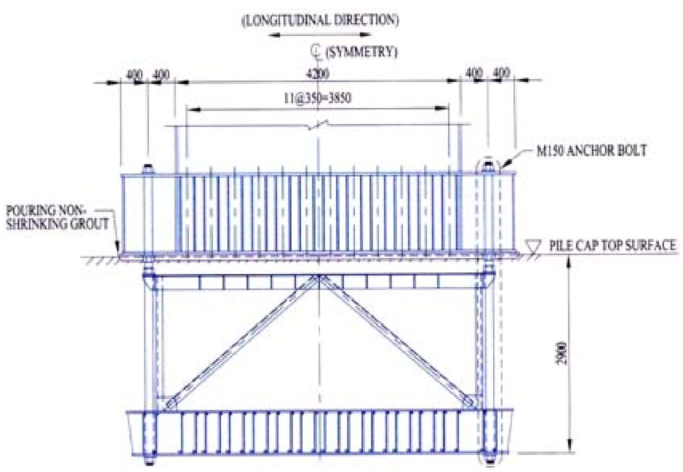


Fig. 6 Anchor frame

Steel pier anchored into foundation by the anchor system consists of column base part, anchor bolts and anchor frame (Fig. 6). The anchor system embeded in the pile cap provides the pull out resistances and friction shear resistances with the concrete of the pile cap.

(4) Fatigue Design of Steel Bridge

For fatigue design of steel bridge the regulations given in the Design Specifications of THSRC have been taken into consideration. Fatigue damage of steel bridge is assessed on the basis of the maximum and minimum normal stresses in flanges and maximum and minimum shear stress in webs of the steel sections due to the load combinations of the traffic model with one or two tracks loaded. The resistance of steel members to fatigue has been checked according to the fracture criteria specified by the specification.

4. Special structural analysis for HSR steel bridge

4.1 Track-Structure Interaction Analysis

The track-structure interaction analysis for the steel bridge bas been performed to assess the longitudinal actions transmitted to the carrying structures in accordance with the design specifications of THSRC. It takes into account of the resistance to longitudinal movement of the track and structural stiffness using a 3-dimensional model. The non-linear interaction between track and structure is modelled by means of bilinear horizontal springs connecting the rails to the structure over the entire length of the model.

The load combinations to be considered for computing the relative displacements between decks per the Design Specifications are:

Normal Operation **L+I+LF**

Type II Earthquake **L1+I1+LF1+EQII**

Where

L= Live load (modified UIC train) **I**= Impact

L1=One train live load **LF**=Longitudinal force (traction + braking)

LF1=Longitudinal force one train (braking)

EQII=Type II earthquake (as per the Design Specifications)

A 3-D frame model comprising twenty-five spans (three main spans and 22 adjacent spans) is formulated by using SAP 2000 program. Two rail tracks are connected to the viaduct via aforementioned elastic-perfectly plastic springs and the constraint conditions are enforced between the corresponding joints of the tracks and the viaducts elements such that they have the same vertical deflections at all time. A non-linear time history analysis is carried out to simulate statically applied braking and traction forces and the dynamic earthquake loading. This is carried out by gradually increasing the external loads using a ramp function over a time span much longer than the longest period of the structure.

According to the Design Specifications, the relative displacement between bridge decks should be less than 7 mm for normal operation (calculated value = 2.34 mm) and 25 mm under Type II earthquake (calculated value = 9.81 mm). In addition, the stresses in the continuous welded rails shall not exceed the allowable stresses stated in Table 1.

Table 1 Stresses limit for welded rail

	Ballast Track
Braking, Traction and 20^{o}C Temperature Variation Between Rails and Deck	-72/+92 MPa (Calculated value *= -59/+66 MPa)*
Braking, Type II earthquake and 20^{o}C Temperature Variation Between Rails and Deck	-147/+167 MPa (Calculated value *=-102/+95 MPa)*

The analysis results corresponding to the aforementioned criteria show that DU-108 can meet all the requirements of the track-structural interaction.

4.2 Dynamic Analysis Of Rolling Stock

The dynamic analysis of rolling stock for the steel bridge has been performed to assess the response of the bridge due to the passage of rolling stock depending on the train velocity and the natural frequency of the structure. The range of train speeds calculated within the analysis is from 120 km/h to 350 km/h and only one train need to be considered.

A three-span frame model is generated by using SAP 2000 program. The moving train is represented by a set of moving concentrated loads. A set of auxiliary time dependent functions is used to control the exact time and location where each specific axle load will be applied. The variation of cant induced by the rolling stock can be evaluated in the same analysis by imposing the axle loads eccentrically from the centroid axis of the viaduct.

According to the Design Specifications, the following safety criteria should be met:

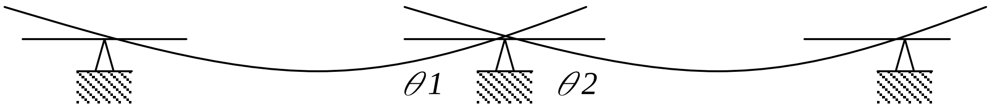
Vertical acceleration of the bridge $\leqq$ 0.35g (calculated value = 0.28g)

Variation of cant by transverse rotation of the bridge $\leqq$ 0.4 mm/m (calculated value = 0.34 mm/m)

Bending rotation of deck end shall not exceed:

$\theta 1 + \theta 2 \leqq 0.008/h$ radian = 0.00192 radian (Calculated value = 0.00032 radian)

Where $\theta 1$ and $\theta 2$ are presented in the following illustration.



h = the distance between the top of rail and the center of the bridge bearing in metre.

The analysis results corresponding to the aforementioned criteria show that DU-108 can meet all the requirements of the structural response due to passage of rolling stock.

5. Conclusion

Because of the lightweight nature of steel structure, the steel bridge of DU-108 is adopted to cross the No. 1 Provincial Highway for 90 m long span, not only to reduced the impact in aesthetic and in heavy traffic to the existing highway, but also to overcome the limited space and headroom available due to the vertical alignment of the HSR line. Various analyses such as track-structure interaction and effects of rolling stock are performed besides the normal design check. The steel structure performs well as the concrete bridges for all the functional requirements of HSR, which specified stringent criteria. The erection work of this steel girders and piers of DU-108 had been done at site as of December 2002, and the whole structure has been completed in April 2003.

Acknowledgement

The authors greatly thank the Evergreen-Shimizu JV, the turnkey contractor, for their support in this project. Special acknowledgement will also be made to the design team members of C296 in MAA for their hard work and enthusiastic in searching solution for all aspects of the project. Encouragement and tolerant from MAA top management are also appreciated.

Reference:

[1] THSRC, *Design Specifications*, Contract Issue, January 2001.

[2] MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, *Seismic Design Specification for Railway Bridges*, 1999.

[3] MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, *Design Specification for Highway Bridges*, 1982.

[4] AASHTO, *Standard Specifications for Highway Bridges*, 1996.

[5] JAPANESE RAILWAY TECHNICAL RESEARCH INSTITUTE *"Railway Structures Design Standard and Commentary – Steel and Composite Structures"*,