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A Draft Design Code for Steel-Concrete Composite Girder in Japan

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Summary

In Japan, a new draft design code for steel-concrete composite girder bridges was elaborated by the committee on composite structures(Chairman:Prof. H. Nakai) in JSCE. The code is mainly for the civil engineering structures such as composite girders in highway bridges and it was written by the format based on a limit state design method.

1. Introduction

The first composite girder bridge(CGB) in Japan was Kanzaki Bridge(Osaka city) of simple span length of 12m. The design was based on German experience and our own experiments. From the practice, The Codes for Design and Construction of Composite Girders on Highway Bridges was published in 1957. Then, the construction method was changed from "propped" to "unpropped" to build more easily. Also, many continuous CGB were constructed. From the late 1960th, non-prestressed continuous CGB had taken the place from difficulties in prestressing at the intermediate support and this type of continuous composite girders was specified in the Japanese Specifications for Design of Highway Bridges(JSHB), 1973. New Kanzaki Bridge is the longest non-prestressed continuous CGB of the span length of 88m. Since 1980th, the construction fever of CGB has decreased remarkably due to deterioration of concrete slabs in composite bridges. Shrinkage of concrete and running of heavy vehicles are the main causes. Recently, by using precast prestressed concrete slabs, the defects can be recovered and CGB are coming to life again.
Under these circumstances, the committee on ultimate strength of composite structures was organized in JSCE in order to prepare a design code for composite structures based on a limit state design method and a draft design code for CGB was elaborated. The paper describes the outline of the code.

2. Application Range and Fundamentals

The draft codes are available to simple composite girders, prestressed or non-prestressed continuous composite girders and composite girders with in-site RC slabs, precast RC slabs and composite decks.

3. Classification of Composite Girders

The composite girders are classified into compact ones and non-compact(slender) ones in relation to the rotation capacity of the cross section. In the compact girder, the composite cross sections can form a full plastic hinge and the ultimate strength of the section can be calculated by the plastic analysis. In the non-compact sections, the ultimate should be limited up to the own when the extreme tension-side fiber stress of steel girder reaches its yielding strength.

The compact section can be determined whether the following equation Eq. (1) is satisfied as well as Eurocode 4 and BS5400,

$$b/t \leqq 9\varepsilon \quad (1)$$

$$d/w \leqq 33\varepsilon/\alpha \quad (2)$$

where, α : ratio of compression zone of web to the web height,
$\varepsilon = \sqrt{(235 / f_y)}$.

The composite action between slab and girder can be expected with ordinary stud arrangement designated in JSHB.

4. Strengths for Materials and Members

Structural steel and concrete should be selected according to the Draft Codes for Steel Structures and the Standard Specification of Concrete of JSCE, respectively.
Headed studs are common for shear connectors. Alternative connectors can be used on pertinent approvals of experiments or research data.

5. Verification for Limit States

Safety for each limit state should be verified with the following fundamental equation Eq. (2) :

$$S_d/R_d \leqq 1.0 \quad (2)$$

where, $R_d = \phi \cdot R(f_d)$, $S_d = S\ (\gamma \cdot F_d)$.

Here, the loading effects F_d and safety factors γ for loads may be adopted of ones designated in the Draft Code for Steel Structures.
The bending moments at internal supports in continuous composite girder may be redistributed by only their 15% from internal supports towards the midspan.

6. Verification for fatigue of studs

The safety for fatigue of studs should be verified with Eq. (3) and (4) [1].

$$\Delta\tau_d/\Delta\tau_R \leqq 1.0 \quad (3)$$

where, $\Delta\tau_d$: the maximum design shearing stress range (Mp),
$\Delta\tau_R$: shearing stress range (Mp),

$$\log \Delta\tau_R = 2.74-0.117 \log N \quad (4)$$

7. Structural details

When the slabs are constructed by in-situ concrete, haunch should be provided at the girder place. The effective breadth of concrete flanges should be determined in accordance with JSHB. When precast slabs are used, a careful attention is necessary for stud arrangement. In the case, group arrangement of plural studs in a position is required and their strength.

Reference

[1] Maeda, Y., Matsui, S. and Hiragi, H. : Effect of Concrete-Placing Direction on Static and Fatigue Strength of Stud Shear Connectors, Tech. Rept. of the Osaka Univ., Vol. 33, No. 1733, pp. 397-406, 1983

A Draft Design Code for Concrete Filled Steel Tubular Columns in Japan

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Summary

In Japan, a new draft design code for concrete filled steel tubular columns was elaborated by the committee on composite structures (Chairman : Prof. H. Nakai) in JSCE. The code is mainly for the civil engineering structures such as piers of highway bridges, compression members of truss and arch bridges, etc. and it was written by the format based on a limit state design method.

1. Introduction

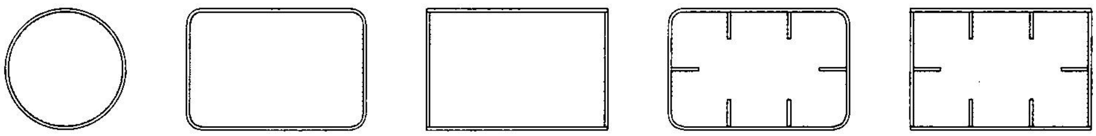
In early 1960th, steel reinforced concrete columns were developed for buildings and concrete filled steel tubular(CFST) columns were applied to electric power transmission towers, in Japan. In the latter half of 1970th, applicability and advantages of CFST columns to the highway bridge piers were investigated through many experimental works. Thereafter, a design recommendation was specified by Hanshin Expressway Public Corporation. In late 1980th, the application was realized for large scale highway bridge piers. Recently, the CFST columns have been attracted special attention as a seismic advantageous type.
Under these circumstances, the committee on ultimate strength of composite structures was organized in JSCE in order to prepare a design code for composite structures based on a limit state design method and a draft design code for CFST columns was elaborated. The paper describes the outline of the code.

2 Application Range and Fundamentals

The types of composite columns are limited to CFST columns as shown in *Fig. 1*, because the columns of bridge piers are remarkably larger than those of buildings. Besides, rigid diaphragms for restraining the filled concrete should be provided at both ends of the column to ensure enough composite action between the filled concrete and steel tube without any shear connectors.
The code can be also applicable for steel contribution factor δ between the following limits, as same as BS5400 and DIN18806.

$$0.2 \leqq \delta \leqq 0.9 \qquad (1)$$

where, δ is the steel contribution factor for compressive strength of composite section.



(1) (2) (3) (1) (2)

a) *Unstiffened type* b) *Stiffened type*

Fig. 1 Typical cross sections of concrete filled steel tubular columns.

3. Basic design strength of composite cross section

3.1 Design strength for axial compression
Ultimate strength of composite column under axial compression P_u can be calculated by using the local buckling strength f_{cuo} based on ECCS buckling curve and design strength of concrete f'_{cd}.

$$P_u = \phi \kappa (f_{cuo} A_s + 0.85 f'_{cd} A_c) \tag{2}$$

where, ϕ is the resistance factor and κ is the reduction factor for general buckling given by cross section of steel member and slenderness ratio of column referred to ECCS buckling curve. A_s and A_c are the cross section areas of steel and concrete, respectively.

3.2 Design strength for flexure
Flexural strength of composite column M_u can be calculated by using the plastic section modulus Z obtained by assuming linear strain distribution at the cross section and neglecting the tension side concrete, because the slip is restrained by rigid end diaphragms. It is experimentally certified that the ultimate bending strength decreases to about 90% of the full plastic moment by local buckling. Therefore, the stress f_{cuo} is used for the maximum strength of plate under compression.

$$M_u = \phi f_{cuo} Z \tag{3}$$

3.3 Local buckling strength of steel tube
For the composite column with rectangular cross section , local buckling strengths of steel tube after hardening of filled-concrete are given for both unstiffened and stiffened type by considering that the plates can deform only outside of the column. For the column with circular cross section, the strengths as steel members may be used.

4. Verification for Ultimate Limit State

The safety of composite columns at ultimate limit state can be verified by using basic design strength given by Eqs.(2) and (3) and the design resultant force for ultimate limit state.
The equations for verifying the composite columns subjected to combined axial compression and flexure are given for uniaxial and biaxial bending using the factor based on *M-N* interaction curves corresponding to δ obtained by many experimental studies. Moreover, the effect of shearing stress due to shearing force and torsional moment to the ultimate strength of composite column is considered by reducing the maximum strength of steel tube based on yielding criterion by Von Mises' hypothesis.

5. Verification for Serviceability Limit State

The permanent deformation due to yielding of surface steel plate must be prevented in order to keep the serviceability and durability of composite columns. Therefore, a verification for working stress is specified in addition to the verification for deformation.

6. Design Details

6.1 Connection of beam-column
When concrete is casted in beam over a half of the flange width from the corner of beam-column joint, shear lag phenomenon hardly occurs in the flange plate of beam. Therefore, the concrete filling range in beams is specified, and the check for shear lag phenomenon is omitted at beam-column joint after hardening concrete.

6.2 Diaphragms
Three-types of diaphragm are specified such as end diaphragms to restrain the slip between filled concrete and steel tube, diaphragms at the beam-column joint to transmit the bending moment of the beam to the column, and intermediate diaphragms to prevent the local buckling of the tube.

Dynamic Response of Curved Composite Cellular Bridges

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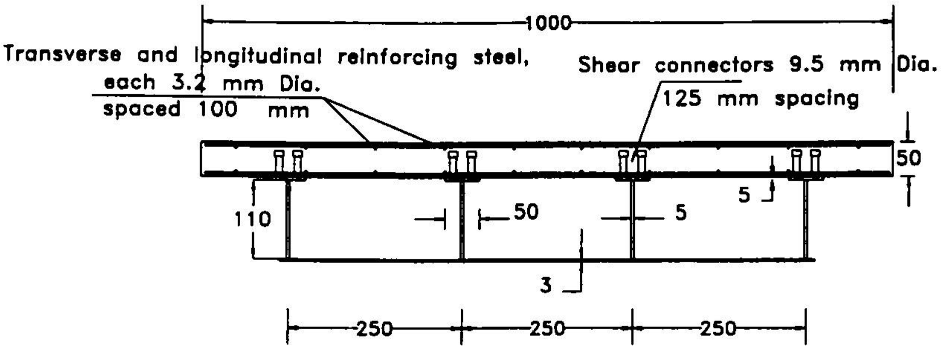
Summary

This paper is a summary of an extensive parametric study , using the finite-element method, in which 120 simply-supported curved composite bridge prototypes are analyzed to evaluate their natural frequencies and mode shapes. The parameters considered in the study are: end-diaphragm thickness, cross-bracing system, degree of curvature, and number of cells. Results from tests on four 1/12 linear-scale simply-supported composite three-cell bridge models of different curvatures are used to substantiate the analytical modelling.

1-Results and conclusion

Dynamic analysis of simply-supported curved multi-cell composite bridges was conducted using the finite-element modelling. This modelling was verified by results from free-vibration testing of four simply-supported composite three-cell bridge models. Figure 1 shows the cross-sectional details of the models. Two diaphragms, 5 mm thick, were placed radially at the extreme end sections. No inner bracings were used in the second model while in the other models, five cross-bracings of rectangular cross-section 13x5 mm were installed in the radial direction, at equal intervals. The span-to-radius ratios were 1.0 in both the first and second models, 0.375 in the third model, and 0.0 in the fourth model. Table 1 summarizes the natural frequencies and mode shapes of the bridge models. Good agreement between the experimental and theoretical findings can be observed. It is observed that the dominant mode of vibration of a curved bridge is a combined flexural and torsional mode, with the bridge frequency decreasing with an increase in the degree of curvature. Also, the presence of cross-bracings enhances the first two natural frequencies in the first model when compared to those in the second model. Figure 2 present the effect of end-diaphragm thickness on the dominant frequency. It is observed that the presence of end-diaphragms enhances the dominant frequency and its mode shapes. Figure 3 shows that for curved bridges, a minimum of three cross-bracings are adequate to enhance the dominant frequency due to increased torsional resistance. Figure 4 shows that the dominant frequency decreases with increase in the curved span as well as in the degree of curvature. In the case of one- and two-lane bridges, when the number of cells is ≤ 3, increasing the number of cells increases the dominant frequency as shown in Figure 5. In the case of bridges with any number of lanes and with number of cells ≥ 3, the change in the number of cells has no significant effect on the dominant frequency. Expressions for the first flexural frequency and hence the dynamic load allowance for this type of bridges has also been deduced and reported elsewhere*.

* Sennah, K. M. 1997. Static and dynamic responses of curved composite concrete deck-steel multi-cell bridges. Ph.D dissertation, University of Windsor, Windsor, Ontario, Canada.



Fig, 1. Cross-sectional details of models

Model No.	L (m)	L/R	First natural frequency (Hz) Experimental	Finite-element	Mode shape	Second natural frequency (Hz) Experimental	Finite-element	Mode shape
1	2.6	1	38	39	LF-TS	125	133	TS
2	2.6	1	36	38	LF-TS	81	91	TS
3	2.6	0.375	44	46	LF-TS	-	138	TS
4	2.6	∞	45	47	LF	-	136	TS
Note: LF: Longitudinal flexure; TS: Symmetric Torsion								

Table 1. Natural frequencies and mode shapes of tested bridge models

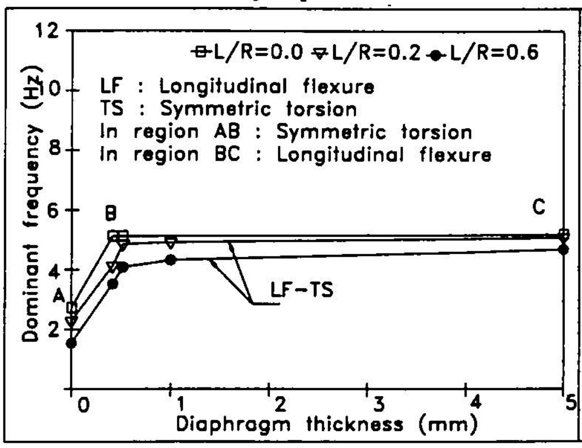


Fig. 2. Effect of end-diaphragm thickness on the dominant frequency of three-lane five-cell bridges

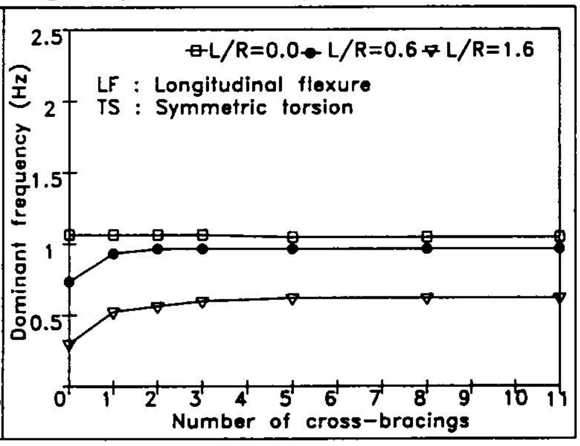


Fig. 3. Effect of cross-bracings on the dominant frequency of two-lane three-cell bridges of span = 100 m

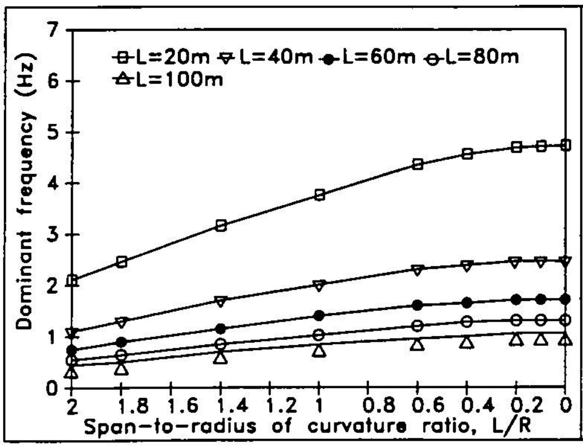


Fig. 4. Effect of curvature on the dominant frequency of two-lane three-cell bridges

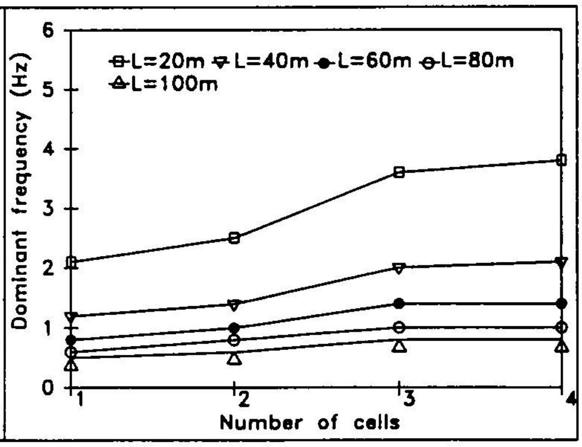


Fig. 5. Effect of number of cells on the dominant frequency of two-lane three-cell bridges with L/R=1.0