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Three-dimensional reinforcement design method and program realization for prestressed concrete box-girder bridges based on a specific spatial lattice grid model

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ABSTRACT

Reinforcement with prestressed steel tendons to prevent cracking is a major design procedure for most concrete box-girder bridges. They provide acceptable performance even though they do not cover all of the principal stress directions of the structural members because ordinary steel bars also play an important role in resisting external loads. However, with the limitations of current design methods, a plane model is always used as a standard for designers for manual performance of the procedure, but it does not provide sufficient information for the reinforcement of easily overlooked parts from the view of integral three-dimensional effects. Design experiences and detailing requirements must therefore be used extensively, leading to either a waste of materials or a greater likelihood of unexpected damage to members. To address this issue, this study proposes a solution for the reinforcement of prestressed concrete box-girder bridges with a new reinforcement method combined with a specific spatial lattice grid model. The model, which finely differentiates the stress sources of the members and especially emphasizes the in-plane stresses of the top and bottom plates, facilitates the establishment of a stress-based reinforcement method to completely consider the three-dimensional effects of prestressed concrete box-girders and provide the necessary information associated with current bridge codes. The method was implemented as the module of a bridge software with an automatic design function. The effectiveness of the method is demonstrated and discussed via numerical examples.

1. Introduction

Prestressed concrete box-girder (PCB) bridges and similar structures [1–8] developed in modern times always adopt a fully prestressed design to prevent the structures from cracking [9–16]. At the beginning, bridge decks were narrow and had a comparatively small traffic volume. As the requirements of transportation increase, box-girders are designed to be wider than ever before [17,18]. Various categories of cracks in such bridges have been discovered, particularly in the rapid construction periods of some countries, and these can significantly hinder the development of similar structures. A typical case was found in Annacis bridge in Canada [17], which is a composite box-girder cable-stayed bridge with a main span of 465 m. Less than 3 months after the bridge was opened to traffic, quantities of cracks appeared in the bridge deck. Among the damage, a second type of crack with a general length of 2 m was discovered near the conjunctions of the cables and the deck at a 45-degree angle to the bridge axis. Cracks with a similar skew in the top and bottom plates of highway PCB bridges have also been found in China [17,18], and in one case, this type of crack accounted for 44% of the total amount at the bottom plate.

In particular, these skew cracks are mostly penetrating cracks that differ significantly from those caused by local effects. Cracks that are accompanied by compressive zones and do not penetrate a plate are always induced by the normal stresses, whereas in-plane principal stresses produce the penetrated cracks of a plate. In essence, the stay-cables at both sides of a cross-section transfer the compressive forces to the middle of the top plate through the in-plane shear stresses, and then cause in-plane principal tensile stresses that reach their peak at the anchorage position and decrease slowly towards the center line of the bridge. These cracks were most probably induced by an overrun of the principal tensile stresses due to unreasonable reinforcement. Once they occur, the skew cracks would stretch along the crack directions until the

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Nomenclature

y_o	distance between the centroidal axis and the upper edge (or y_0)
y_c	depth of the compressive zone
x	distance between the compressive zone centroid and the upper edge
A_{s1}	steel bar area in the upper edge
A_{s2}	steel bar area in the lower edge
e_o	eccentricity of the axial force (or e_0)
$e_1(e_2)$	distance between the steel bar centroid in the upper (lower) edge and the cross-sectional upper edge
f'_{sd}	compressive design strength of steel bars
f_{sd}	tensile design strength of steel bars
f_{cd}	compressive design strength of concrete
f'_{s1}	steel bar stress in the compressive zone, take f'_{sd} in the limit state
f_{s2}	steel bar stress in the tensile zone, f_{sd} in the limit state
f_{ch}	concrete stress in the compressive zone, take f_{cd} in the limit state
F_{ys}	concrete cross-section area of the compressive zone
M	bending moment acting on the centroidal axis
N	axial force acting on the centroidal axis
γ_0	important coefficient of the bridge structure
M_2	resistant bending moment of the axial force with respect to the A_{s2} resultant point
ξ	relative compressive zone height
ξ_b	relative-boundary compressive zone height
h_0	cross-sectional effective height, equals to e_2
η	magnifying coefficient of eccentricity of the axial force for eccentricity compression members
$\rho'_{\min}$	minimum reinforcement rate
$e(e')$	distance between the axial force action point and the A_{s2} (A_{s1}) resultant point
E_s	modulus of elasticity of the ordinary steel bar
E_c	modulus of elasticity of the concrete
ξ_{cu}	ultimate compressive strain of the concrete
β	height coefficient of the compressive zone
σ_1	maximum principal tensile stress
f_t	uniaxial tensile strength of the concrete
f_c	uniaxial compressive strength of the concrete
$\sigma(\tau)$	normal (shear) stress
$c(\mu)$	cohesion (friction coefficient) of the material
b	width of the web plate
N_k	cross-sectional axial resistance of the reinforced concrete
a'_s	distance between the steel bar resultant point of the cross-sectional compressive zone and the compressive zone edge
h_{0i}	distance between the cross-sectional centroid of the longitudinal i-th layer steel bars and the edge subject to the larger compression
σ_{0i}	stress of the longitudinal i-th layer steel bars
ξ_{cp1}	concrete compressive strain at the A_{p1} resultant point when the member is damaged
σ'_{p1}	prestressed tendon stress at the A_{p1} resultant point
σ'_{p1c}	concrete stress at the A_{p1} resultant point induced by the prestressing
n_p	elasticity modulus ratio of the prestressed tendon to the concrete in the compressive zone
f'_{p1y}	compressive strength design value of the prestressed tendons
ξ'	distribution coefficient for the maximum shear force design value allocated to the concrete and stirrups, $\xi' \geq 0.6$
f_{sk}	tensile strength standard value of stirrups
A_{sv}	total cross-sectional area of the stirrups at the same concrete cross-section
$\sigma_{zl}(\sigma_{zv})$	principal tensile (compressive) stress
$\sigma_x(\sigma_y)$	normal stress in the orthogonal direction of the micro-unit
$A_{sx}(A_{sy})$	steel bar area in the X (Y) direction
θ	angle between the principal stress direction and the Y direction steel bars
S	length of the oblique section
$\mu_{sx}(\mu_{sy})$	reinforcement rate in the in-plate orthogonal X (Y) direction
σ_g	steel bar control stress
f_{tk}	standard value of axial tensile strength
$\mu_{\min}$	minimum shear resistance reinforcement rate per unit length

structure splits and will even introduce staggered deformations. The stay-cables or prestressed tendons may then fail to effectively transfer the designed forces to satisfy the structure's safety requirements. Thus, the damage was not merely a problem of negligence in calculation, but a matter of method in terms of modeling and reinforcement.

Current bridge codes [19–22] always use a two-dimensional (2D) beam model [23–29] as a standard for manual reinforcement. In this model, the bending-shear stresses of web plates and the bending stresses at the upper and lower edges of the PCB cross-section are considered to compose the major reinforcement information. However, this model is merely applicable to the design of a thin-walled narrow beam, but it cannot objectively reflect the states of a wide bridge structure because it ignores the effects of spatial deformation. Although an independent transverse frame analysis [30,31] may be performed as a complement, it cannot represent the integral three-dimensional (3D) effects of the structure. With such a model, a general fully prestressed design cannot cover all principal stress directions of the structural members, particularly the in-plane ones in the top and bottom plates. Therefore, the ordinary steel bars in these positions assume greater importance than local strengthening or crack-resistance alone [32,33]. However, very few scholars have emphasized the important role of ordinary steel bars in resisting the integral effects of a PCB bridge. Instead, when faced with structural development, design experiences and detailing requirements were therefore used extensively, leading to either a waste of materials or a greater likelihood of unexpected damage to members.

For this purpose, Zhao [34,35] performed 11 T-section model tests to verify the shear resistance function of orthogonal steel bar grids and found that all longitudinal steel bars can contribute to the shear strength of a concrete beam. This finding indicates that a feasible 3D model of the PCB that can fulfill the ordinary steel bar design against integral effects is necessary. However, the analytical results from a solid model are always mixed with quantities of local stress concentration, which cannot provide direct reinforcement due to the current beam-oriented code systems. A beam element with 14-degrees of freedom was presented in the literature [36] to consider the warping effects for the design of a PCB precast segmental bridge, and a similar method was used for the amplification factors of the shear stresses in three PCB bridges [18]. Although a single-beam element with multiple degrees of freedom considers the 3D effects of a box-girder to some extent, it cannot help to detail a transverse distribution of the mechanical characteristics [37–40], and the web plate's shear force must be calculated based on the assumption that the closed cross-section is taken as an open one, which greatly reduces the simulation accuracy. Sun [41,42] used the well-known Hambly planar grid model [43] to consider some transverse allocation of shear forces for simulation of a wide bridge deck. However, this model was constructed by cutting a closed cross-section into several open ones, which introduces errors because the