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The Safety Assessment and Reinforcement of Interchange Ramp Bridge

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The interchange ramp bridge in study is a continuous pre-stressed concrete box beam bridge. A comparative study of calculated rupture conditions of beam structure under different assumptions and the practically tested rupture conditions illustrates that the side-span beam structure seriously lacks of internal existing prestressing force and fails to meet the regulation requirements, owing to which it needs reinforcement. For the purpose of better visual effects, decisions have been made to adopt external prestressting approach inside the box girder. Numerical analysis shows after reinforcement the structural carrying capacity and partial stress-strain both can meet regulation requirements. The comparison of the static loading tests before and after reinforcement further indicates that the employment of external prestressing approach notably enhances the overall stiffness of the beams and fulfills the design purpose of reinforcement of the beam structures.

1. Introduction

The interchange ramp bridge is located at the southern ring of the Guangzhou City belt road which is also a part of the national trunk line. Being 17.3 m wide(Xu and Zhang 2008), it is a continuous pre-stressed concrete box beam bridge, with four lanes, span being 3×25m, continuous pre-stressed concrete box girder being 1.65m high, the mid-span section web being 0.4 thick, bottom plate being 0.22m, top plate being 0.25m, the pivot point of the section web at the foot of the girder being 0.6m, side pivot being 0.5m.

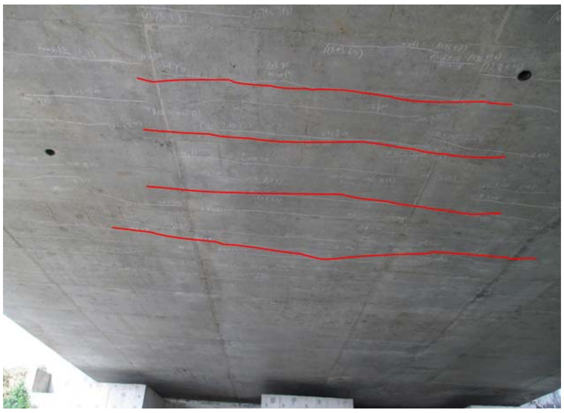

Figure 1: Transverse cracks on the bottom

The appearance inspection and static loading tests show that: there exists many transverse cracks at the bottom of the 1st and the 9th span continuous box beam with the largest crack being 0.2mm wide (Figure 1) (Yepes et al. 2015);a small amount of the cracks even extend to the web and presenting a L-shaped spatial crack(Figure 2) (Yang et al. 2015). The interval between bottom plate transverse cracks is about 20~30cm and the cracks distribute all the span(Pircher et al. 2011). With corresponding loading test on appropriate working condition(Valipour et al. 2015), the stiffness of the mid-span section of the 1st span fail to meet the

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design requirement. During the static loading test the cracks on the box girder is observed to open and close, which indicates that the cracks are under active state(De Brito J et al. 1997)]. To guarantee the beam structure safety, the operating safety and the structure durability, effective measurements are required to be adopted to reinforce the continuous pre-stressed concrete box girder to improve the carrying capacity and the safety stock(Sagara and Pane 2015).

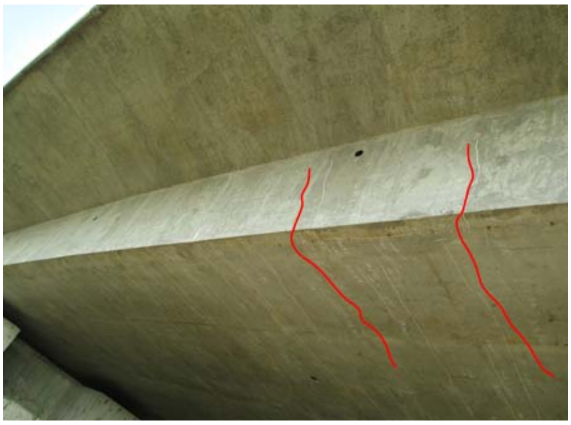

Figure 2: Transverse cracks extend to the web

2. Bridge reinforcement measurements

2.1 Main bridge checking conclusions

Calculate the bridge crack situations of the operating tail-hole (the 1st span and the 9th span) under different calculation assumptions (Table 1) and go on to compare the results with the practically tested crack situations (Table 2-Table 3) to determine the internal existing prestressing force of the 1st span and the 9th span. The results can be seen as reference regarding structure reinforcement(Peyton et al. 2012).

Table 1: Summary table of assumptions calculation

Assumption calculations	steel beam area (compared to the design drawing)	Steel beam tensile stress (compared to the design drawing)
Assumption 1	100%	50%
Assumption 2	100%	0%
Assumption 3	0%	0%
Assumption 4	50%	100%
Assumption 5	25%	100%

An comparative analysis reaches the conclusion that assumption two and assumption five might be right(Natário et al. 2015), the positive stress of which doesn't meet the regulation requirements for normal usage in a short time(Figure 3). The flexural resistance under assumption two meets the regulation requirements, while assumption five doesn't.(Figure 4) (K and A 2003, Caterino et al. 2014).

Table 2: Summary of the crack width comparison

Project	Crack width(mm)	Crack distribution (meters away from 0#)
Practical test	Maximum 0.02	-
Assumption 1	0.0009~0.033	10~20
Assumption 2	0.19~0.47	2.75~20
Assumption 3	0.4~1.29	2.75~20
Assumption 4	0.007~0.042	10~20
Assumption 5	0.02~0.21	2.75~20

Table 3: Summary of static loading test comparison

	Results	Assumption 2	Assumption 4	Assumption 5
Variation of crack width(mm)	0.033	0.1284	0.0203	0.0463

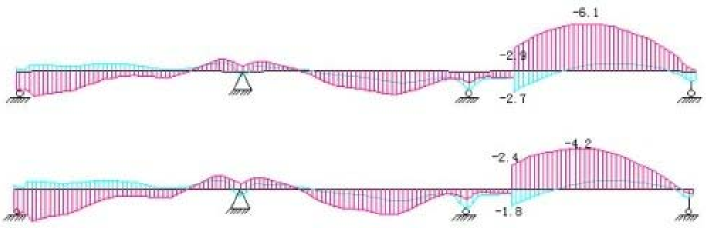

Figure 3: Positive-stress of normal usage for a short time

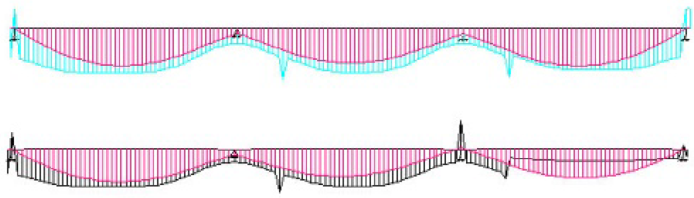

Figure 4: Ultimate flexural resistance

2.2 Structure reinforcement goal and measurement

According to the above bridge checking results and the construction condition requirements, to guarantee the applicability and durability of the structure the purpose of the reinforcement design is: after reinforcement, the structures can meet A-level standard of the original design(Maas et al. 2012).

Based on the above reinforcement purpose, measurements have been taken at corresponding place of the 3×25 two span(the 1st span, the 9th span) web of the stated interchange ramp bridge(Ramnavas et al. 2015). For the purpose of bettering the visual effects after reinforcement, all the reinforcements are carried out inside the box girder(de Brito and Branco 1997). For each span impose 1860Mpa standard value tensile strength, 12 bundles of 15.2mm nominal diameter filling epoxy coating steel hinges with excellent corrosive resistance and 15 hinges for each bundle(Reza et al. 2014). The steel cables are placed near the web, anchored at the anchorage area of the beams. To avoid too much stress on partial area, the beams with the anchorage are placed separately along the bridge, with the interval between two anchorage area being 150cm(Charuchaimontri et al. 2008).

For the convenience of future external examination and maintenance, a human-hole is designed at appropriate place of the bottom plate. Measurements of partial reinforcement and temporally enclosure of the human-hole are carried out.

2.3 Steps of reinforcement construction

(1) Design checking human-hole and bonding human-hole plate at the bottom.

First clean the concrete surface and unveil the new appearance. Probe the steel location at the bottom plate of the box girder and punch a hole. Plant anchorage bolt. Then punch the same hole in the steel plate and install the steel plate and fasten the worms. Fill in and compress the sticky steel glue and carry out steel plate anti-corrosive operation(Yoon et al. 2014).

(2) Plant the steel cables

Carve a groove where the new and the old concretes of the steel cable anchorage beams meet according to the construction drawing(Nelson and Fam 2014). Combine with the original construction design drawing and detect the layout of the common steel and cables of the interface between anchorage beam and the original concrete with rebar detector. Mark appropriate places and punch holes to plant the steel cables. Strap the anchorage beam steel and position the pre-stressed steel cables.

(3) Pour the steel cables to anchor beams

Design moulds to pour steel cable anchorage concrete by reference of the construction drawing and homogenously brush(or spray) the interface glue to the interface of the new and the old concretes before pouring. Watch out for the embedded parts of the pre-stressed steel cable anchor.

(4) Tension the external pre-stressed steel cables

The stiffness of the steel cable anchorage concrete reaches 90% of the standard value and tension the concrete for at least 7 days. Then start to tension the external pre-stressed steel cables and the box girder webs of both sides begin to tension the steel cables symmetrically at the same time.

3. Check the reinforcement effects

The external pre-stressed strength stated above is determined by the envelope calculation of the assumption two and assumption five. The below presents the carrying capacity and partial stress checking conditions under different assumptions by means of the external pre-stressed reinforcement mentioned in this article.

3.1 Assumption 2: existing pre-stressed force being 0% of the original design
Determine the below working condition by taking the box girder condition into consideration:
(1) Medium-loading test of the largest positive bending moment and deflection with the least favorable loading at the middle span;
(2) Eccentric-loading test of the largest positive bending moment and deflection with the least favorable loading at the middle span;
(3) Medium-loading test of the largest positive bending moment and deflection of B-B section with the least favorable loading at the side span;
(4) Eccentric-loading test of the largest positive bending moment and deflection of B-B section with the least favorable loading at the side span;
The positive section flexural resistance and shear strength calculating results of the external steel cable reinforcement beam under the lasting carrying capacity ultimate state are presented in Figure 5 and Figure 6.

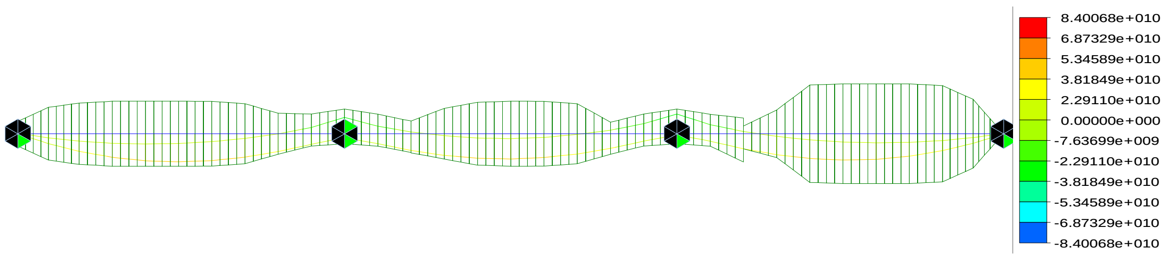

Figure 5: Envelope of structure flexural resistance(kN.m)

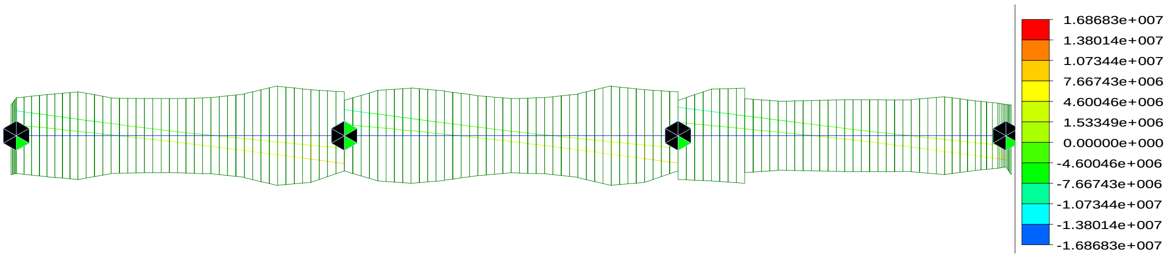

Figure 6: Envelope of structure shear strength(kN)

Figure 5 and Figure 6 indicate that the structure carrying capacity of assumption 2 meets the regulation requirement.

3.2 Assumption 5: existing pre-stressed force being 25% of the original design
Determine the below working condition by taking the box girder condition into consideration:
(1) Medium-loading test of the largest positive bending moment and deflection with the least favorable loading at the middle span;
(2) Eccentric-loading test of the largest positive bending moment and deflection with the least favorable loading at the middle span;
(3) Medium-loading test of the largest positive bending moment and deflection of B-B section with the least favorable loading at the side span;
(4) Eccentric-loading test of the largest positive bending moment and deflection of B-B section with the least favorable loading at the side span;

Figure 7 and Figure 8 present the positive section flexural resistance and shear strength calculating results of the external steel cable reinforcement beams under lasting carrying capacity ultimate state.

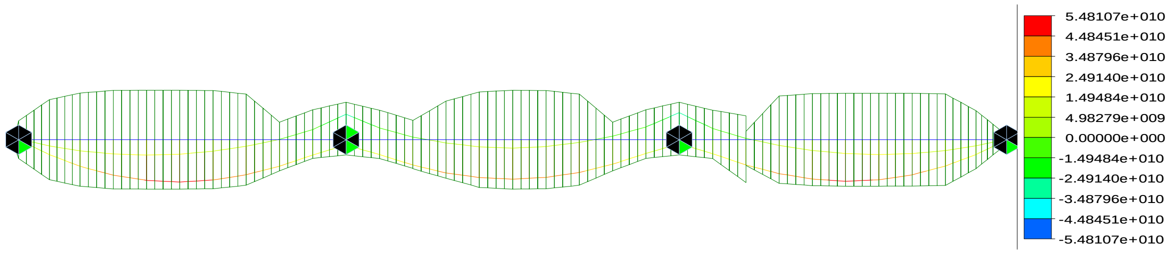

Figure 7: Envelope of structure flexural resistance (Unit: kN.m)

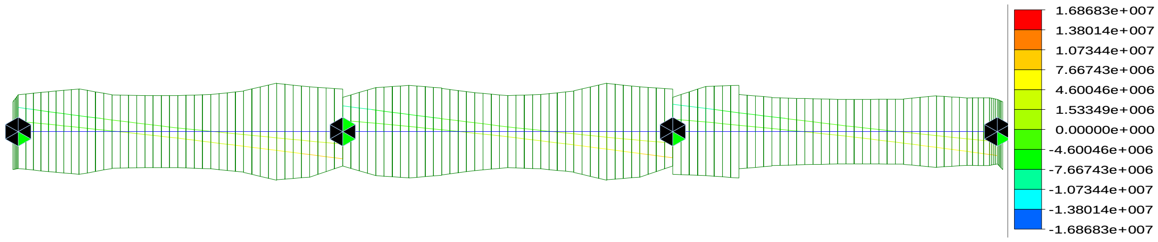

Figure 8: Envelope of structure shear strength (Unit: kN)

Figure 7 and Figure 8 show that both of the structure carrying capacity of assumption five can meet the requirement.

4. Analysis of partial anchorage beams

Create a model to calculate by means of a finite element software ANSYS(Aggelis and Shiotani 2007), with the concrete beam being solid65 unit, the entire beam being 27m long, the side beam of pre-stressed steel cable tension end face being 20m long, the size of steel cable tensile anchorage beam along the bridge being 1.5m, the height being the same as the girder. The prestressing model is a homogeneous distribution model imposing on the column of the practical anchorage point, with the steel cable being 15Φs15.2. Both ends of the model beam are strapped in longitudinal, lateral and vertical directions. The model comprises of 160,000 node points and 720,000 units.

It can be seen from Figure 8 2.4 MPa stress along the bridge is calculated at the interface between the front beam anchorage surface and the bottom plate. Owing to the stress diffusion effect of the front beam anchorage pre-stressed force, little tension stress is calculated at the interface between the backward beam anchorage surface and the bottom plate.

5. Conclusion

Many assumptions are made regarding the insufficient carrying capacity of a interchange ramp bridge and numerical mimic analysis of these assumptions determines the internal existing pre-stressed force of the bridge structure. Reinforce the bridge by means of external pre-stressed force. A comparison of the twice static test results before and after reinforcement indicates that the employment of external pre-stressed force reinforcement has effectively enhanced the overall bridge stiffness and fulfilled the design purpose of the reinforcement.

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