A REAL CASE STUDY ABOUT EVALUATION OF THE EXISTING SITUATION OF A POST TENSIONED SINGLE SPAN BOX GIRDER BRIDGE

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Abstract This paper covers evaluation of a constructed bridge, experiencing some deformation of concrete section before service stage. Bridge is a single span-post-tensioned – box girder bridge casted on scaffolding, constructed in Northern Iraq. Box section has a second order parabolic arch shape in longitudinal direction. After removal of scaffolding, excessive vertical displacement occurred. Location and alignment of the cracks indicated a horizontal shear transfer problem. This thesis is supported with the calculations appeared in the paper. Furthermore, a separate model is prepared and the results are compared with the real displacements.

1 Introduction

The properties of the bridge construction site (span lengths, pier heights), engineering demands (superstructure widths) and environment determine the type of the bridging system. In old times, arch geometry has been widely used especially for single span bridges supported on freely movable abutments. [1] With developing post-tension technology, it is possible to design posttensioned box girder bridge. In the early 1960’s, the California Department of Transportation (Caltrans) studied on box girder bridges with post-tensioning. [2] In these type of bridges the depth of cross section can vary on the longitudinal bridge direction. On the abutments the depth of the box is higher and it decreases in order to get the thinnest depth located in the middle of the span. In this paper, a constructed single span posttensioned box girder bridge experiencing some problems before the service stage is investigated.

The bridge is constructed as a mono cell box- single span bridge with total length of 64.8 meters and two twin decks. Box section has a variable depth with a second order parabolic geometric shape. The depth is 4 meters at the abutments and in mid span it is 2 meters (Figure 1, 2 and 3). There are seven internal diaphragms along the bridge span. Two of them are at the ends which have 2 meter thickness and solid in shape. Other five diaphragms are placed in the span with equal intervals and have thickness of 0.5 meter with an access space. Superstructure is supported by two elastomeric bearings located under each end diaphragm. In each web, there are 20 groups of post-tensioning cables each having 22 tendons. The bridge was constructed by full staging method on scaffolding. For checking the design and detect reasons of problems, a separate analysis is performed to investigate and understand the problem in design and if there is in construction.

2 Bridge Geometry

2.1 Cross-sections:

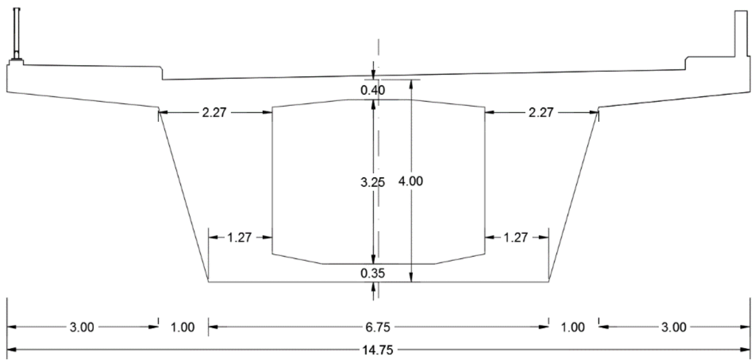

Fig. 1. Cross-section of superstructure at abutments

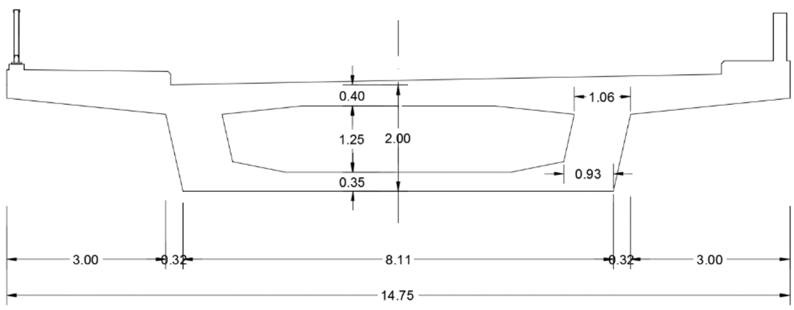

Fig. 2. Cross-section of superstructure at mid-span

2.2 Plan and Profile view

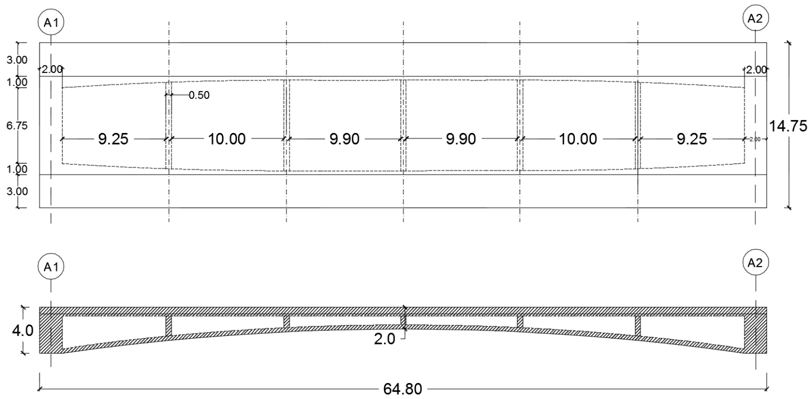

Fig. 3. Plan and profile view of the bridge

3 Presenting Problems Occurred in Bridge

Excessive Displacement on Midspan

According to the records provided by the construction control engineers, after scaffoldings are removed, deflection at mid-span started to occur in a decreasing trend and finally reached to 70 cm vertical deflection. Deflections are more than theoretical deflections since, deflections about 15-16 cm was predicted for this stage in the light of the structural analysis of the bridge

Beside vertical deflection, excessive horizontal cracks are observed at the intersection of top slab and web (Figure 4). The cracks are especially remarkable at the abutment zone between last two diaphragm walls. The concrete cover spalled and concrete at the interface section were crushed. Vertical reinforcements were bended along the crushed zone. It seems that reinforcements were not be able to develop the yield strength under horizontal shear forces because they are bended along the crushed zone.

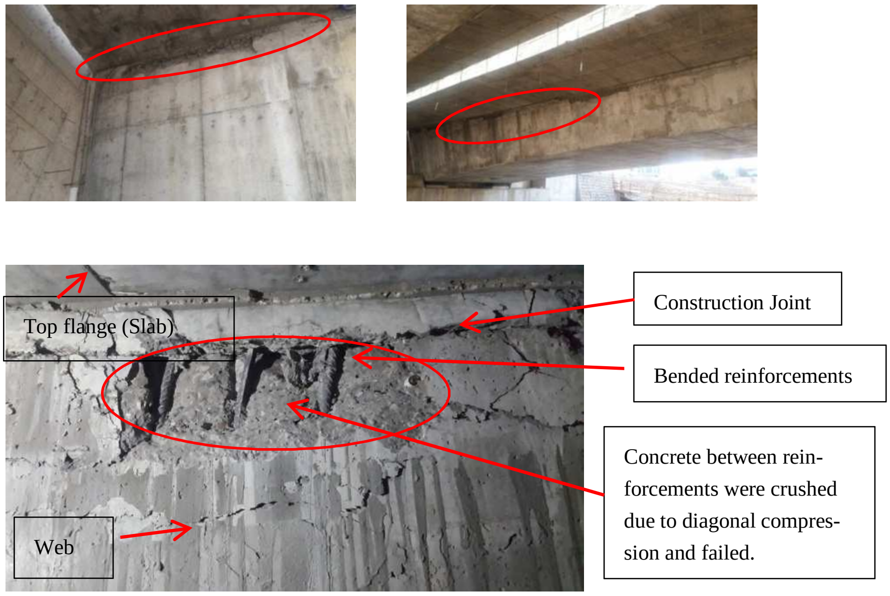

Fig. 4. Horizontal cracks at web-slab intersection

4 Detailed Calculations for Investigating the Problem

After such indications of a horizontal shear strength problem; a detailed calculation about the horizontal shear transfer was performed. For this purpose, AASHTO LRFD specification is taken as reference According to code;

$V_{ri} = \phi V_{ni}$ and $V_{ri} \geq V_{ui}$ (5.8.4.1-1) and (5.8.4.1-2)

Where;
V_{ni}= nominal interface shear resistance
V_{ui} = factored shear force due to load based on the applicable strength and extreme event load combinations. (Horizontal shear force)
ϕ = resistance factor
The nominal shear resistance, V_{ni} shall be taken as:

$V_{ni} = cA_{cv} + \mu (A_{vf}f_y + P_c)$ (5.8.4.1-3) and also Maximum resistance shall be less or equal to the lesser of,

$V_{ni} = K_1 f_c A_{cv}$ or

$V_{ni} = K_2 A_{cv}$ in which $A_{cv} = b_{vi} * L_{vi}$

A_{cv}= area of concrete considered to be engaged in interface shear transfer (mm^2)
A_{vf}= area of interface shear reinforcement crossing the shear plane within the area A_{cv} (mm^2)
b_{vi}= interface width considered to be engaged in shear transfer (mm)
L_{vi} = interface length considered to be engaged in shear transfer (mm)
c= cohesion factor (MPa)
μ= friction factor
f_y = yield stress of reinforcement but design value not to exceed 60 ksi
P_c = permanent net compressive force normal to the shear plane; if force is tensile, P_c=0
$f_{c'}$= specified 28 day compressive strength of the weaker concrete on either side of the interface (MPa)
K_1= fraction of concrete strength available to resist interface shear
K_2= limiting interface shear resistance

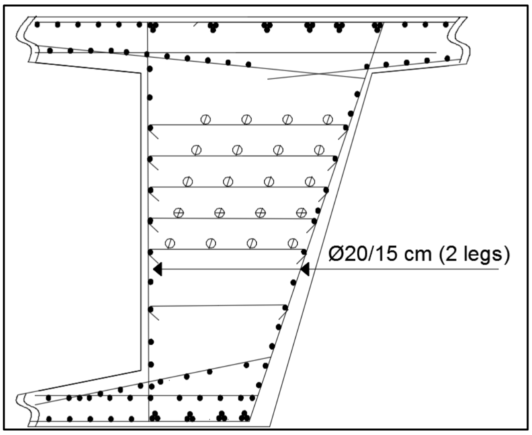

Fig. 5. Shear reinforcement at anchorage zone and interface between web and slab

It is observed that at the site, construction joint surfaces are not intentionally roughened which affects the capacity (Figure 5). According to standards, the following parameters which are listed in Table 1 should be used for smooth surfaces (or not intentionally roughened surfaces).

Table 1. Parameters for not intentionally roughened surfaces (AASHTO LRFD –Ch.5.8.4.3)

c (cohesion factor) (in MPa)	0.52
μ (friction factor)	0.6
K_1 (fraction of concrete strength available to resist interface shear)	0.2
K_2 (limiting interface shear resistance) (in MPa)	5.5

Table 2. Calculations of horizontal shear check

A_{cv} (mm^2)	4540000	for 2 webs/1 m
A_{vf} (mm^2)	8373	2 webs have totally 4 legs of ϕ20/150 mm
ϕV_{ni} (kN)	4023	nominal interface shear resistance
V_{ri} (kN)	4024	factored shear force from Method-1
V_{ri} (kN)	4620	factored shear force from Method-2
$K_1 f_c A_{cv}$ (kN)	36320	> V_{ri} OK
$K_2 A_{cv}$ (kN)	24970	> V_{ri} OK

Factored horizontal shear force can be calculated by using two approaches:

Method-1: Classical Elastic Method

$V_h = VQ/I$

V =factored vertical shear force at section
Q = first moment of area of the portion above the interface with respect to the neutral axis of section
I = moment of inertia of the composite cross section

Method-2: Simplified Elastic Beam Behavior

In this method, the procedure in AASHTO LRFD C.5.8.4.2 is used.

$M_{u2} = M_1 + V_1 \Delta l$

$C_{u2} = M_{u2}/d_v$

$C_{u2} = M_1/d_v + V_1 \Delta l / d_v$

$C_1 = M_1/d_v$

$V_h = C_{u2} - C_1$

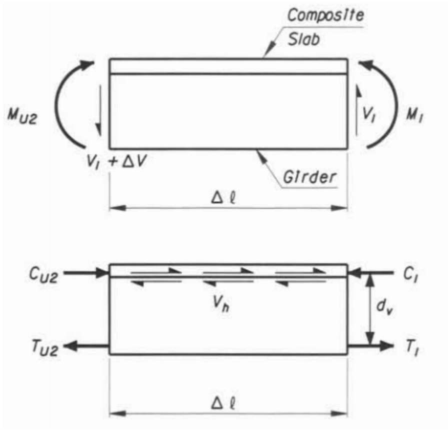

Fig. 6. Free body diagram

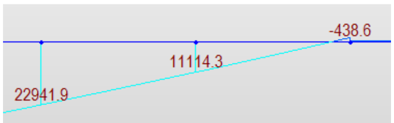

Fig. 7. Moment diagram of the bridge under dead and superimposed dead loads

According to moment diagram M_{u2}=22942 kN.m, M_1=11114 kN.m, C_{u2}= 22942/(0.8*4)= 7169 kN and C_1= 11114/(0.8*4)= 3473 kN. So V_h= C_{u2}- C_1=3696 kN. If this load is multiplied by 1.25 (load factor for service case), then V_{hi} = 1.25*3696= 4620 kN/m

The calculated horizontal shear strength of the bridge, considering only the self- weight and superimposed dead load and excluding the wearing surface and live loads (since the bridge is not opened to service yet) is smaller than the applied load. Furthermore, live load effect and asphalt on the superstructure should be included to total factored vertical shear force for the reinforcement design of the section. Therefore, it is observed that the provided reinforcement is inadequate.

5 Reasons of Excessive Deformations and Vertical Displacement

During the site visit it was observed that vertical reinforcement at the construction joint interface on top was bended along the crushed zone. This observation indicates that provided vertical reinforcements could not be able to develop yielding under shear forces. After movement between top slab and webs were initiated, reinforcements could not resist the horizontal shear forces and crushing of concrete and bending of reinforcement occurred. That excessive deformation resulted in loss of the composite behavior between top slab and rest of the box section which caused reduction in the inertia of the section. At this stage, horizontal shear forces increased since V_h value in the formula $V_h = VQ/I$ increases and it exceeded capacity of the joint. After losing the composite behavior, rigidity of the whole box girder dropped seriously causing too large vertical displacements. There was another reverse effect caused by the post tension on cracked section which is variation of the center of the gravity of the section approaching to bottom slab and that created positive primary moments acting on the same direction with vertical loads it also increased the vertical displacements until a new balanced case was hold.

6 Comparing the deflection between computer model and site measurements

Bridge was modeled according to the documents (drawings, calculation report etc.) gathered from the client. Midas Civil program was used for this purpose (Figure 8). This program can consider time dependent effects and construction stages can be formed to see the steps followed on site. Firstly, model was prepared by confirming to the bridge geometry. Then boundary conditions were introduced to model. After that, load were applied to model and the results were studied. Since compositeness was lost due to insufficiency of horizontal reinforcement, in computer model, slab and webs were separated in abutment region to model this new condition.

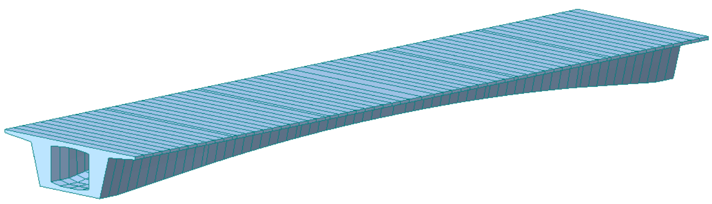

Fig. 8. 3-D computer model of bridge

Table 3. Parameters used in computer model

Parameters used in computer model	
Superstructure:	fc= 40 MPa
Unit Weight*:*	Υ = 25 kN/m^3 for reinforced concrete Υ = 24 kN/m^3 for asphalt cover.
Modulus of Elasticity:	E = $0.043 * \Upsilon_c^{1.5} * \sqrt{f_c}$ = 31980 MPa for concrete
	E = 200000 MPa for strands and steel
Other Data:	fy= 420 MPa for steel fu= 1860 MPa for prestressing tendons
	fy= 1674 MPa for prestressing tendons
Prestressing Tendons	20 pieces tendon group in each web, where each group holds 22 tendons
Elastomeric Pads	1200x1200x35 cm

Table 4. Loads on model

Loads on model	
1. Dead Load	Self-weight is automatically considered by the program.
2. Superimposed Dead Loads	Sidewalks, parapets, guardrail and leveling concrete are considered
3. Post-Tensioning Loads	Loads exerted from posttension cables.

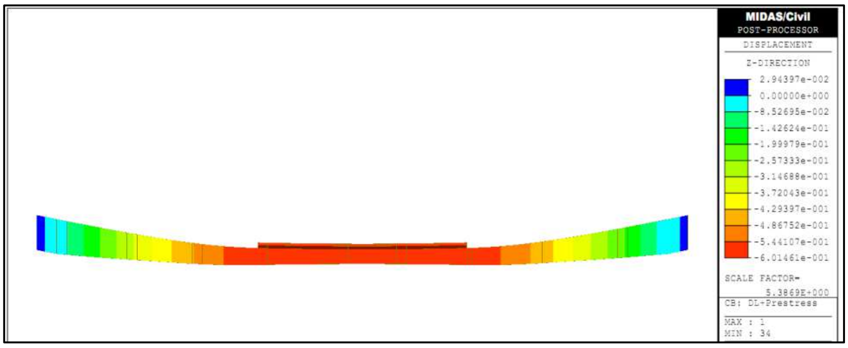

Fig. 9. Vertical deflection of bridge under posttensioning and dead loads.

As can be seen from Figure 9, 60 cm vertical deflection occurred in midspan which is compatible with the site measurement which was about 63 cm.

6 Conclusion

Horizontal cracks which were observed at site between slab and web interface indicated a horizontal shear problem at first glance. Detailed calculations about this interface showed a capacity lack since only two legs shear reinforcement were placed at anchorage locations. Smooth surface at construction joint also played a significant role in capacity loss. Since horizontal shear capacity was exceeded, box-girder had lost its composite behavior which leaded cracks in the slab-web interface. In addition, due to lowering of the center of gravity of the section, post-tensioning cables played a negative role especially at end zones. When all these effects were combined, excessive vertical deflection in mid-span occurred. This excessive deflection is compatible with the computer model results..

7 References

1) Karaesmen, E. (2015). *Ardgermeli Beton ve Yeni Çözümler* (1st ed., Vol. 1, Ser. 22858). Istanbul, Türkiye: Lord
2) Post-Tensioned Box Girder Design Manual, Rep. No. FHWA-HIF-15-016 (2015)
3) AASHTO LRFD Design Specifications 4th Edition 2007