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Experimental study on transverse behavior of concrete box girder bridges with strutted wing slabs

Viet-Hung Tran ^a, Thi-Nguyet-Hang Nguyen ^a, Phuc-Thanh Trinh ^a, Viet-Hung Cu ^{a,*}

^a Faculty of Bridges and Roads, Hanoi University of Civil Engineering, 55 Giai Phong Street, Hanoi, Vietnam

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ABSTRACT

In 2023, the first box girder bridge with strutted wing slabs in Vietnam was put into use in the Ring Road II Viaduct Project in Hanoi. As this is an innovative type of traditional box girder bridges and completely new in Vietnam, prior to the commencement of the project, an experimental program was conducted to check the construction technology and obtain valuable experimental data for design and further numerical analyses. A full-scale girder segment with a width of 18.7 m, a length of 6 m and height of 2.5 m, incorporating steel pipe struts, was fabricated. Three distinct load cases were conducted to investigate the transverse behavior of strutted wing slabs, steel pipe struts and connections under the most unfavorable load conditions at Service Limit State. This paper presents a detailed experimental program with test results. It is shown that with applied loads varying from 1.2 to 1.5 times the service load, both the deck slab and steel pipe struts still exhibited linear manner, maintaining sufficient load-bearing capacity. However, with increased applied load in Load Case 2, cracks occurred along the construction joint region, indicating that attention to construction joint quality in real projects is needed to pay great attention to ensure the integrity of structures. Based on the conducted experiment, some recommendations are proposed for future implementation of such bridge typologies.

1 Introduction

Box girder bridges with strutted wing slabs have been utilized in Europe since the 1970s, representing an innovative approach to the traditional box girder bridge design. In this configuration, wide cantilever slabs, known as wing slabs (typically exceeding 4 m in length), are supported by diagonal struts made from various materials such as concrete, steel pipes, or advanced composites, etc. The incorporation of wide wing slabs not only provides increased space for traffic flow on the bridge but also allows for the utilization of struts to support them, thereby minimizing the bottom width and reducing the self-weight of the box girder. Consequently, the dimensions of substructures, including pier columns, abutments, and foundations, can be decreased [1, 2]. As exemplified by the Incheon Bridge in Korea, the use of struts has led to a 15% reduction in superstructure material usage and a 50% reduction in substructure material compared to

* Corresponding author. Tel.: +84 0913328789

E-mail address: hungcv@huce.edu.vn

traditional box girder alternatives. Moreover, the slender girders and pier columns not only contribute to cost savings but also enhance the aesthetic appeal of the bridge. Additionally, optimizing pier size creates more space for traffic beneath the bridge, particularly crucial in urban areas where land scarcity is a pressing concern. Furthermore, as highlighted by Shushkewich [3], strutted box girder bridges offer the advantage of future expansion to accommodate increased traffic volumes. Initially designed as a two-lane bridge, these structures can be widened to three or four lanes when necessary [3, 4]. To date, numerous strutted box girder bridges have been constructed in Europe, including notable examples such as the Kochertal Viaduct in Germany, Rogerville Viaduct in France, Piedrafita Viaduct in Spain, and the WA 19 Viaduct in Portland.

While the advantages mentioned above should ideally propel the popularity and widespread adoption of strutted box girder bridges beyond Europe, their utilization remains limited in other regions. Japan saw the introduction of strutted box girders in the early 2000s, specifically for highway bridges along the New Tomei Expressway [1]. It wasn't until 2009 that the first strutted box girder bridge in Korea, the Incheon Bridge, was put into use. Presently, Vietnam joins the ranks with the completion of the Ring Road II viaduct in 2003, marking the first bridge incorporating strutted wing slabs in Vietnam.

It is believed that the limited use of strutted box girders is partly due to the lack of published research and available examples to help engineers understand behavior of this innovative type of structures. Indeed, while there has been numerous research on behavior of traditional box girder bridges [2-15], the number of studies on strutted box girder bridge has been limited [1, 16-22]. Only several superstructure designs of real projects were introduced in previous studies (Uchimaki Viaduct, Japan [22, 23], Katsurashima Viaduct [16], Incheon Bridge, Korea [21]). Experimental investigations on this type of bridge are extremely insufficient. To the best knowledge of the authors, only one experimental study on a box girder bridge with concrete-filled fiber reinforced polymer (FRP) struts has been found in the literature [21]. The lack of published studies on behavior of box girder bridges with strutted wing slabs made it more difficult for engineers to understand and comfortably apply them in practice. Since the structural characteristics of this innovative bridge have not been thoroughly understood, engineers would encounter dilemmas in designing steps. One of these could be how to design deck slabs with wing parts strutted in transverse direction.

To address this research gap, an extensive experimental program was conducted on a full-scale box girder bridge incorporating steel pipe struts, with three primary objectives: (1) Verification of proposed construction technology: the program aimed to validate the proposed construction methodology prior to its application in Vietnam's Ring Road II Viaduct, ensuring its feasibility and efficacy in real-world scenarios; (2) Investigation of transverse behavior of deck slab supported using diagonal steel struts: a focal point of the study was to analyze the transverse behavior of this innovative box girder bridge type, with an emphasis on the responses of the wing slab, steel pipe struts, and joints under service loads; (3) Provision of valuable experimental data: the experimental program sought to generate valuable data for future design and facilitate subsequent numerical analyses, thereby contributing to the advancement of engineering knowledge in this domain. Details of the experimental program and results from the tests are presented and discussed in the paper. Such experimental results can serve as valuable reference for professionals engaged in similar projects, aiding in their decision-making processes and contributing to the overall advancement of bridge engineering practices.

2 Experimental program

2.1 Overview

To check construction technology and investigate the transverse behavior of a strutted box girder bridge under service load, a full-scale prestressed concrete box girder employing diagonal steel pipe struts was cast and three load cases were conducted. The study aimed to examine the behavior of the wing slab, the steel pipe struts, and the connections between the steel pipe and concrete under service loads in Service Limit States (SLS). According to AASHTO LRFD Specification, SLS is the state of design beyond which structural components lose operational serviceability [24]. Under SLS, the tested specimen is required to satisfy deflection, stress, and crack width (if any) within allowable limits. Exceeding SLS limits may necessitate repair or replacement, while repeated exceedance can lead to deterioration and eventual collapse of structures. Fig. 1 depicts the overall cross-section of the strutted box girder and Table 1 provides details of the three load cases in the experimental program.

In the transverse direction, the deck slab was prestressed using prestressing tendons. Six tendons were evenly spaced at intervals of 1 m along the specimen's length. Each tendon comprised four 7-wire low relaxation prestressing strands with a strand diameter of 15.2 mm. These prestressing tendons were pre-stressed to 75% of their ultimate capacity (1860 MPa), resulting in a jacking stress of 1395 MPa. While Fig.4 presents prestressing tendons' arrangement in both transverse and longitudinal directions, properties of material used in this program are summarized in Table 2.

Table 2 – Material properties

Material	Type used	Yield strength, f_y (MPa)	Ultimate strength, f_u (MPa)	Elastic Modulus, E (GPa)
Concrete	Compressive strength, $f'_c = 47.86$ MPa*	N.A	N.A	38.5*
Reinforcing bars	CB400-V	400	570	200
Prestressing tendons	Four 7-wire/low relaxation strands	1670	1860	197
Steel pipe struts	Grade 345	345	450	200
Studs	Grade 345	345	450	200

* Defined by tests using 150×300 mm cylinder specimens

2.3 Specimen fabrication

The specimen was cast in two separate stages. Initially, the core segment, with a width of 12 m as depicted in Fig. 1, was cast. Two days later, the formwork of the core segment was removed, and the steel pipe struts were erected. Subsequently, the casting of the wing slab took place, 25 days after the core segment was cast. Once the wing slab achieved sufficient strength (3 days after casting), transverse tendons were prestressed, and the concrete deck began to undergo post-tensioning, transforming into a prestressed slab. It's important to note that this construction procedure followed a fabrication plan for a box girder bridge with strutted wing slabs in Japan [25]. In this project, the concrete core segment was precast, and the cast-in-place wing slab was then produced at the construction site. Fig. 5 illustrates the procedure for fabricating the girder segment.



a) Core segment was cast



b) Steel pipe struts were erected



a) Reinforcement installation for the wing slab



b) After the whole wing slab was cast

Fig. 5 – Specimen fabrication procedure: (a) → (b) → (c) → (d)

2.4 Inspecting the girder segment during fabrication and before testing

The specimen underwent close monitoring throughout the fabrication process to detect any defects or cracks that would have arisen. Inspection procedures involved both visual examination and the utilization of specialized crack monitoring equipment. Generally, no major cracks were found in the girder segment during fabrication, except for some shrinkage cracks. However, a few cracks, with a maximum width of about 0.6 mm and a length of 15 cm, were observed in the concrete at the lower end of the steel struts, as depicted in Fig. 6. These cracks likely developed due to the forces exerted during the tightening of bolts. Additionally, it was noted that the two steel plates in connections, one embedded in the concrete and the other at the lower end of the steel struts, were not fully aligned. Further investigation into the concrete strength in these areas was conducted using Rebound Hammer Tests, as illustrated in Fig. 7. The results showed that the concrete at several locations did not meet the required strength.



Fig. 6 - Cracks at the lower end of the struts during fabrication



Fig. 7 - Measuring concrete strength using Rebound Hammer Test

The cracks observed at the connection between the core segment and the lower end of the struts during construction suggested that, in practical applications, it's advisable not to position the steel plates too close to the bottom of the girder. This adjustment can help minimize the likelihood of such cracks occurring. Additionally, extending the length of the studs could effectively reduce the bond stress between the studs and the concrete. Moreover, it's essential to meticulously design the detailing of the connection between the concrete and the steel pipe struts. This includes avoiding the arrangement of dense reinforcement, as it can hinder the proper filling of concrete, resulting in poor concrete strength in these areas. In addition, it is necessary to employ appropriate casting methods to ensure the solidity and strength of concrete at both ends of the struts. Most importantly, precise technical instructions for tightening high-strength bolts are necessary. It is crucial to clearly state that no bolt should be tightened first to fix the strut, but rather, they should be tightened evenly and sequentially according to the procedure.

2.5 Test configuration & instrumentation

2.5.1 Test configuration and load calculations

To investigate the transverse behavior of a strutted box girder bridge, with a focus on the performance of the wing slabs and steel pipe struts under Service Limit State conditions, three load cases were planned and executed. The test configurations for Load Cases 1 to 3 are illustrated in Fig. 8 (a) to Fig. 8 (c), respectively.

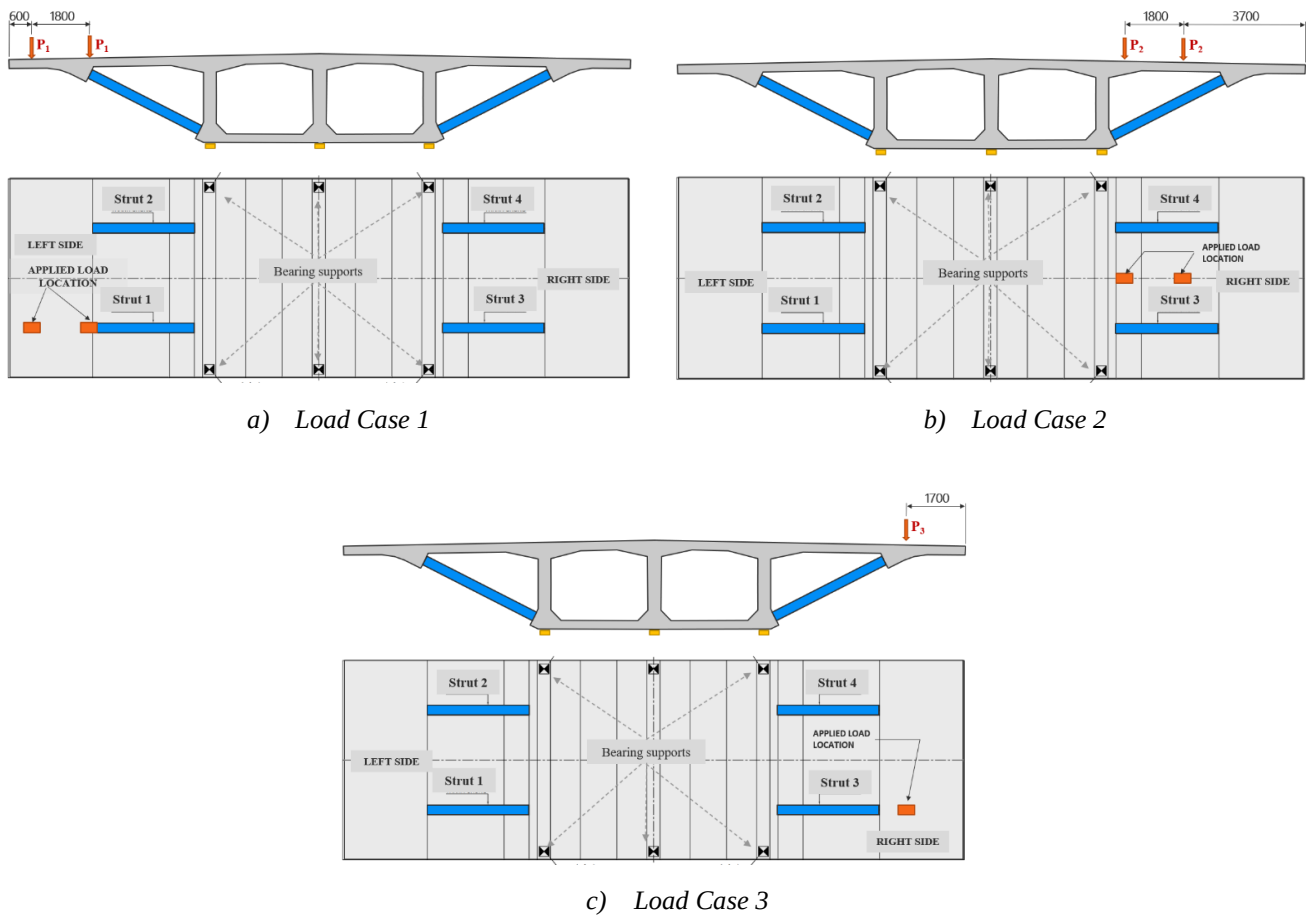


Fig. 8 – Test configuration

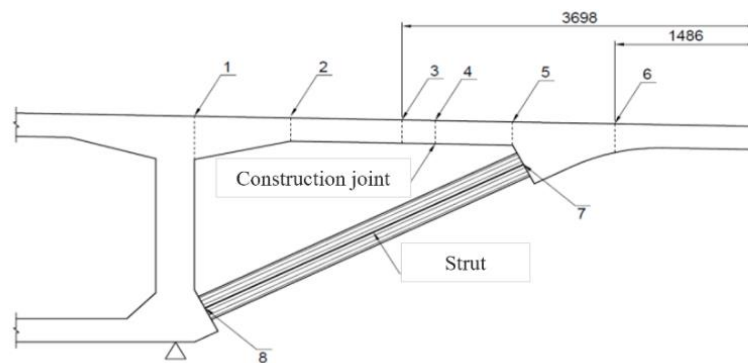


Fig. 9 – Critical sections in the wing slabs

It's essential to note that the Vietnamese design code - 22TCN 272-05 [26], which was completely based on the AASHTO LRFD Bridge Specifications (1998) [27], served as the basis for designing the girder segment. Consequently, the locations of applied loads in Load Case 1 and Load Case 2 were planned, taking into account the potential positions of the two axles of the HL-93 design truck (Fig. 10), as specified in 22TCN 272-05 and the AASHTO LRFD specifications. These positions were deliberately selected to induce the most adverse effects on the wing slabs. Particularly, Load Case 1 was arranged to generate the maximum negative moment in the left wing slab (as depicted in Section 6 shown in Fig. 9), while Load Case 2 was positioned to produce the maximum positive moment in the right-wing slab (as indicated in Section 3 in Fig. 9). Detailed computations yielded values of 120 kN for P_1 and P_2 in the first two load cases. This value corresponded to the load effects imposed by the design truck, with 72.5 kN allocated to each axle of the HL93 design truck

(Fig. 10), augmented by a factor of 1.25 to account for vehicular dynamic load allowance (IM) as specified in the two design codes, along with the additional dead load from wearing surfaces and barrier curbs applied on the deck slab.

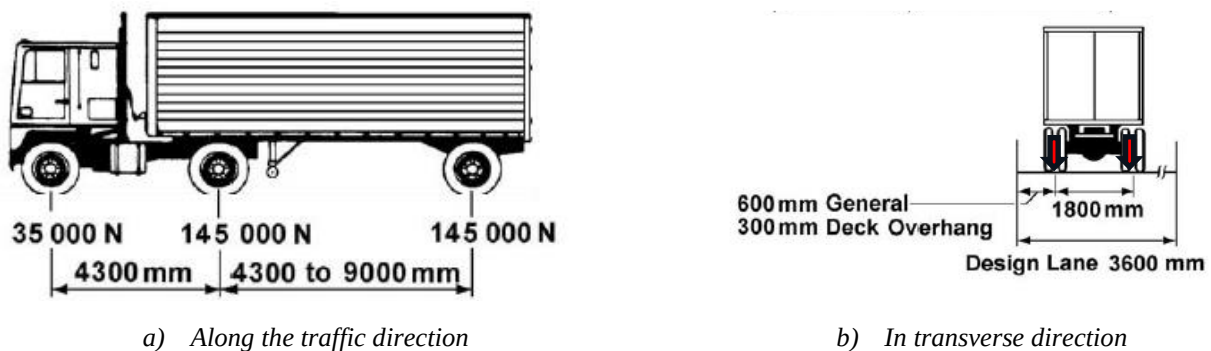


Fig. 10 – Characteristic of the design truck in HL93 (live load) according to AASHTO LRFD [24]

On the other hand, Load Case 3 (illustrated in Fig. 8(c)) was uniquely conducted with a single concentrated load positioned at the upper end of Strut 3. This setup was specifically designed to scrutinize the compression and flexural characteristics of the strut under service load conditions. To produce an equivalent load effect in the investigated strut under Service Limit State, a value of 270 kN for P3 representing additional dead load and design truck of the live load HL-93 was calculated and proposed in the test.

2.5.2 Test setup and instrumentation

The setup for the real site test is illustrated in Fig. 11. Essentially, the specimen was placed on a reinforced concrete base strengthened through six bearing supports. The locations of these supports between the specimen and the base are depicted in Fig. 8(a) to Fig. 8(c). In all three tests, the load was applied to the specimen using a hydraulic jack with a maximum capacity of 150 tons and a reaction system as shown in Fig. 11(a). The increasing loads during the tests were monitored using a load cell positioned directly below the jack, as illustrated in Fig. 11(b). Since a hydraulic jack was employed, the tests were under load control. Load increments between two steps in all tests were managed to not exceed 10% of their maximum proposed loads.

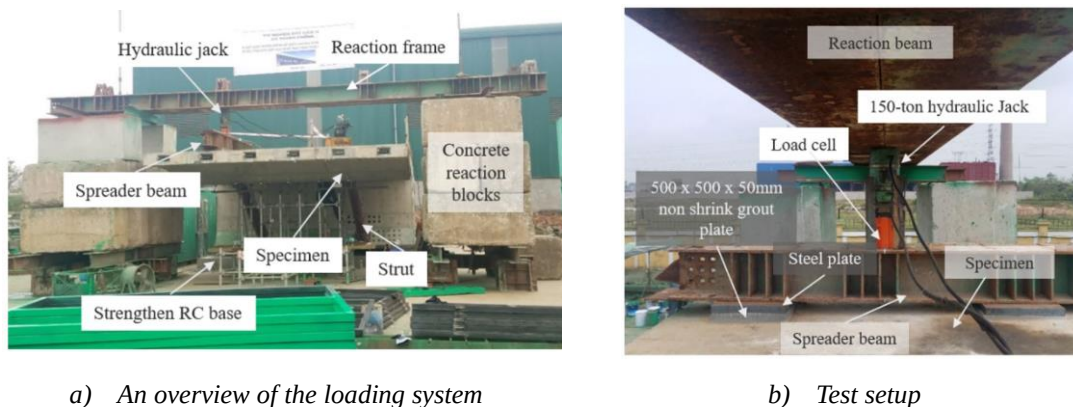


Fig. 11 – Real site test setup (typically shown for Load case 1)

Displacements of the specimen at planned locations during the tests were measured using dial indicators. There were 12 dial indicators arranged on each wing slab side, resulting in 24 different measurement locations. Additionally, the settlement of the six bearing supports was monitored using another six dial indicators. Fig. 12 displays the dial indicator arrangement on the specimen.

Apart from the load cell and dial indicators, strain gauges were attached to the wing slabs and the steel pipe struts to measure developed strains and, consequently, developed stresses in these planned locations. The locations of the attached

strain gauges are presented in Fig. 13 to Fig. 15. As shown in Fig. 13, concrete strain gauges were placed on the top and bottom soffits of the wing slab at several critical locations such as at the construction joint (C2, C6), and the location with the maximum negative moment (C4, C8), etc. Additionally, steel strain gauges were attached to the four struts at their upper and lower ends, as well as their middle, as shown in Fig. 14. It should be noted that instrumentation is only shown for one half of the specimen (on the right side), and the arrangement of indicators and strain gauges on the other half (the left side) of the girder was symmetrically performed. Strain gauges were also attached to reinforcement at conjunctions between the concrete slab and steel struts (Fig. 15). From the compatible deformation assumption, strain in concrete, and then local stress in concrete at the joints are determined.

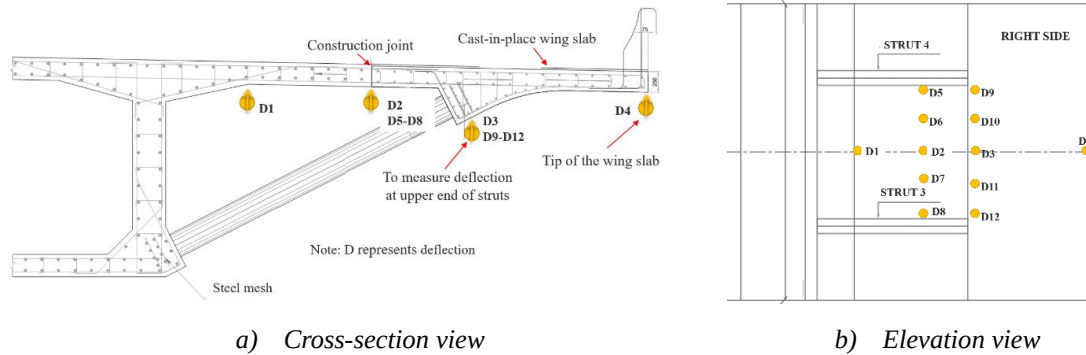


Fig. 12 – Dial indicator arrangement for displacement measurement (shown for the right side)

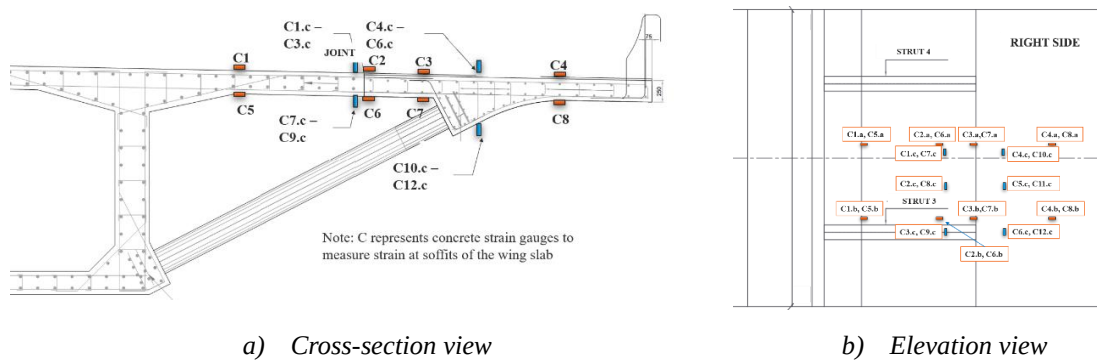


Fig. 13 – Concrete strain gauge arrangement (shown on the right side)

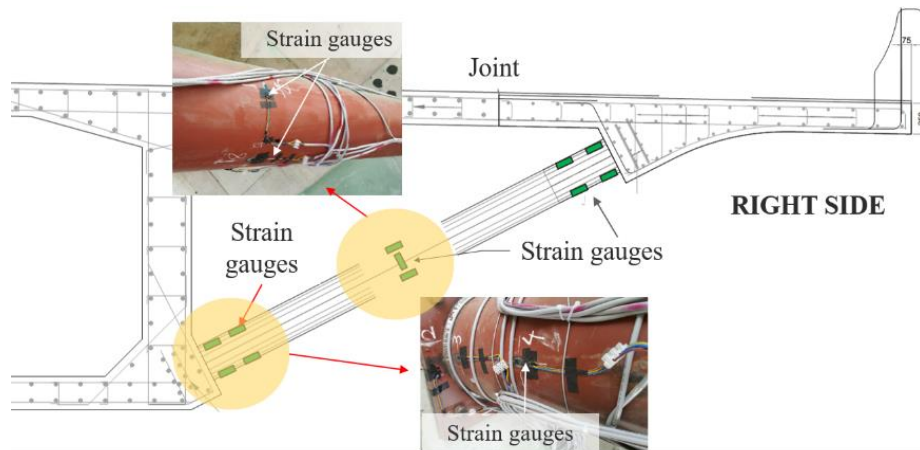


Fig. 14 – Strain gauges on the steel pipe struts

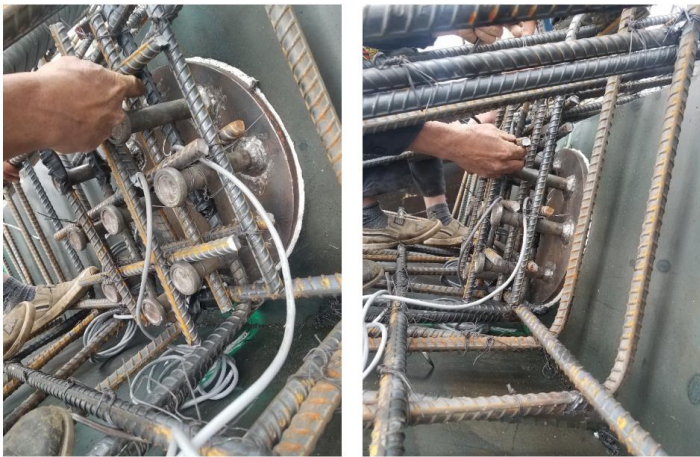


Fig. 15 – Installing strain gauges in connections between Strut 3 and concrete

3 Experimental results and discussions

The responses of the deck slab to applied loads in the three load cases will be presented and discussed in terms of deflections and stresses at critical locations in the wing slab. Allowable stresses in the concrete, as per the 22TCN 272-05 Standard [26], are calculated and shown in Table 3. On the other hand, the behavior of the investigated steel pipe struts will be analyzed using developed stresses, axial force, and moment in the struts.

It is important to note that the stress results presented in Section 3 are values calculated from measured strains in concrete or steel, multiplied by their respective elastic modulus. This approach is reasonable as the girder segment was operated under Service Limit State conditions, and linear behavior would be expected. Furthermore, the axial force and moment of the investigated struts were calculated from measured strains at the soffits of the struts, as shown in Fig. 16.

Table 3 – Allowable concrete stress in Service Limit State according to 22TCN 272-05 Standard

	Equation	Values (MPa)
Allowable compressive stress	$0.4 f'_c$	-18
Allowable tensile stress	$0.5 \sqrt{f'_c}$	3.35

f'_c : design compressive strength of concrete; tension (+); compression (-)

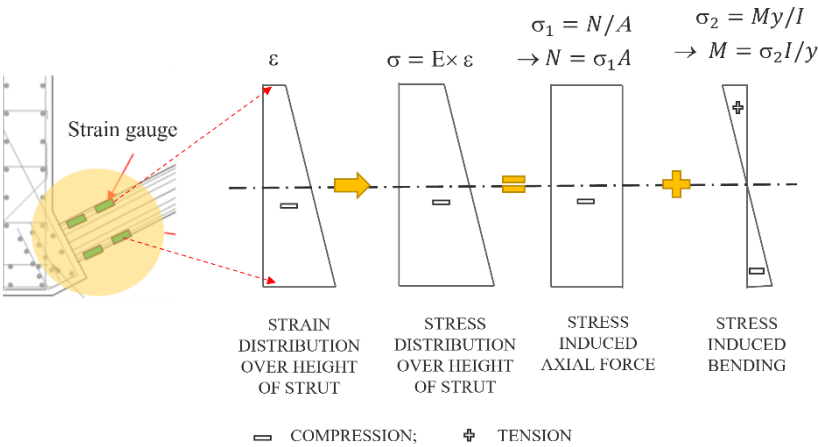


Fig. 16 – Calculations of axial force and moment from measured strut strains

3.1 Structural response under Load Case 1

As shown in Sec. 2.5, in Load Case 1, two concentrated loads were imposed on the left-wing slab. The proposed value for P_1 was 120 kN, resulting in a total superimposed load of 240 kN on the specimen. However, during the test, it was decided to increase P_1 from 120 to 180 kN (1.5 times of the proposed value) to examine the behavior of the left-wing slab and Struts 1 and 2 in case an overload occurs.

3.1.1 Response of the deck slab - Load Case 1

Fig. 17 displays load-displacement curves determined from the load cell and dial indicators installed prior to the test. Notably, the deflections depicted in Fig. 17 represent values after mitigating the influence of local settlements in the bearing supports. In fact, during the experiment, it was observed that the local settlement values following several initial load increments were negligible (less than 0.01 mm), confirming the solidity and stability of both the supports and the base under the applied loads.

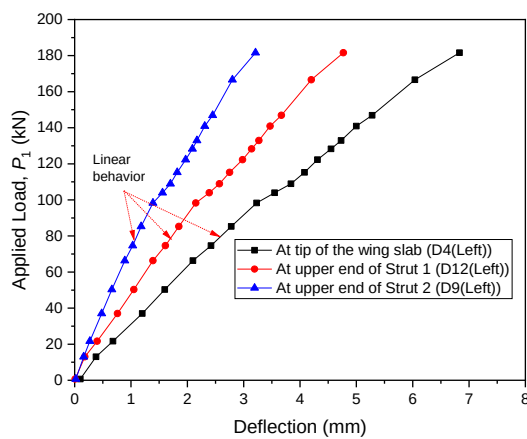


Fig. 17 – Load-deflection curves – Load Case 1

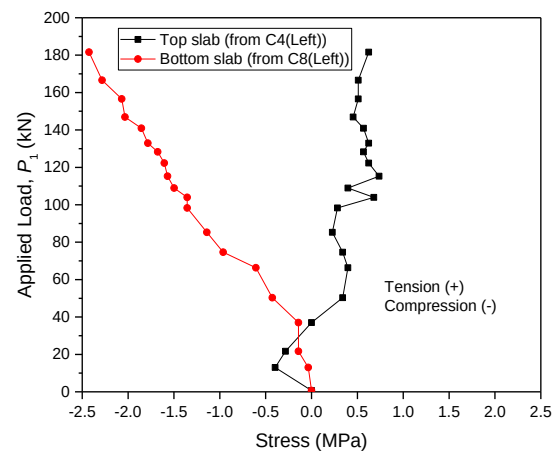


Fig. 18 – Stresses at top and bottom slab at Section 6 – Load Case 1

Fig. 17 illustrates an anticipated trend, with maximum displacement observed at the tip of the left-wing slab, where loads were applied. Furthermore, the deflection at the upper end of Strut 1, closer to the imposed load locations, exceeded that at the upper end of Strut 2, situated further from the applied load locations.

The maximum deflection recorded at the left-wing slab was 4.31 mm when P_1 was approximately 120 kN, representing the calculated total load at the Service Limit State. With an increase in the imposed load to 180 kN, the maximum deflection at the tip rose to 6.83 mm. Notably, even when P_1 was increased to 1.5 times the proposed load, the load-displacement relationship remained linear, with no significant cracks observed. This observation is consistent with the stress results shown in Fig. 18 which shows that maximum compressive stress (at bottom slab) and tensile stresses (at top slab) at Section 6 (the critical section for this load configuration as shown in Fig. 9) were -2.43 and 0.74 MPa, respectively. These values are both significantly smaller than allowable stresses in Service Limit State shown in Table 3.

3.1.2 Response of the steel pipe strut – Load Case 1

The stress distribution and internal forces within Strut 1 under increasing applied loads are illustrated in Fig. 19 and Fig. 20, respectively. In Fig. 19, a nearly linear relationship is evident between the applied load and the induced stress in Strut 1. Notably, the maximum compressive stress observed in Strut 1 was -73.62 MPa when the actual applied load, P_1 , reached 1.5 times the proposed load (180 kN compared to the initial proposed load of 120 kN). This value significantly falls below the yield strength of the steel pipe struts, which is given as 345 MPa in Table 2, indicating that Strut 1 remained within its linear deformation range. However, as can be noticed in Fig. 19, while stresses at the two opposite surfaces (top and bottom) of the upper end of Strut 1 were almost identical, there were significant differences between these stresses at

the lower end of Strut 1. This implies that during the load testing, the lower end of Strut 1 was rotated, and moment at this location occurred. To further elucidate this behavior, internal forces including axial force and moment were calculated from measured strains (the procedure shown in Fig. 16) and presented in Fig. 20. Fig. 20 shows that moment occurred at the lower end of the strut and increased with the applied load. This experiment data demonstrated that with the occurrence of moment in Strut 1, when designing the steel pipe struts, these struts should be treated as “beam elements” instead of “truss elements”.

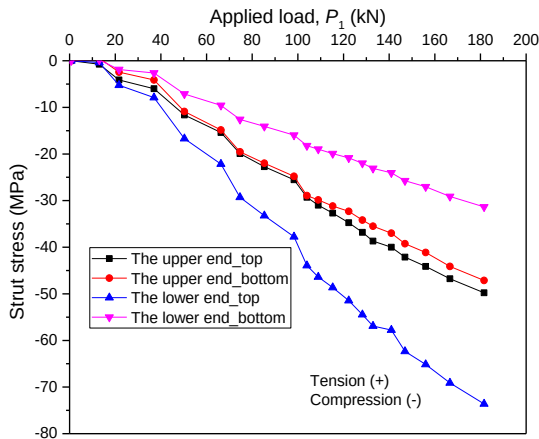


Fig. 19 - Load vs. developed stress in Strut 1 under Load Case 1

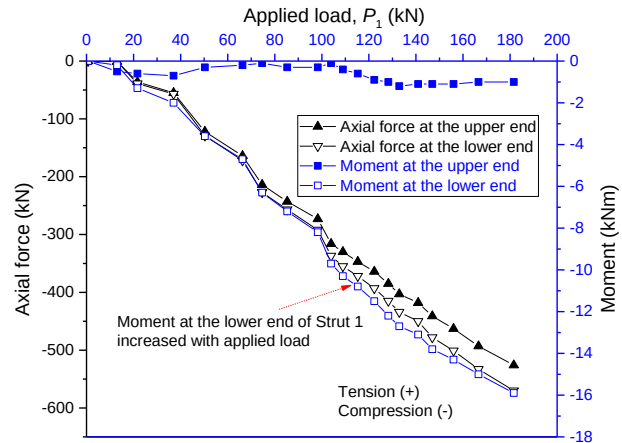


Fig. 20 - Load vs. developed internal forces in Strut 1 under Load Case 1

During the Load Case 1 test, local stresses in the concrete at the connections between Strut 1 and the edge beam (at the upper end), as well as the lower flange of the girder segment (at the lower end), were calculated using installed strain gauges. However, experimental data revealed that at the maximum load applied, the stresses in the concrete were small, less than 0.9 MPa in both compression and tension, and thus significantly smaller than the allowable concrete stresses at the Service Limit State. The testing of Load Case 1 generally indicated that despite the applied load being greater than the calculated service load (1.5 times the maximum service load), the deck slab and the steel pipe struts remained in the linear stage. No significant cracks or abnormal phenomena in response to the structure were observed, demonstrating that the design was safe for the structure working under Load Case 1 conditions.

3.2 Structural response under Load Case 2

Load Case 2 was conducted to produce the maximum positive moment of the right-wing slab (Section 3 in Fig. 9). As can be seen in Fig. 8(b), two concentrated loads representing two axles of the design truck HL93 were imposed on the right-wing slab. The value of P_2 was calculated, resulting in a value of 120 kN to produce an equivalent positive moment at Section 3 in case all the service loads (including dead load and live load) acting on the bridge applied. However, similar to Load Case 1, during the test, the girder segment was loaded at an actual load greater than the proposed value. In this case, P_2 was increased up to 144 kN (1.2 times the planned value) to investigate behavior of the structure beyond Service Limit State. It should be noted that the experimental program presented in this study included a total of three tests, of which Load Case 1 was conducted first. Thereafter, Load Cases 2 and 3 were subsequently performed. While the imposed load was applied on the left-wing slab in Load Case 1, it was arranged on the right-wing slab in Load Cases 2 & 3. Therefore, the increase of P_2 compared to the proposed value was only 1.2 times (different from 1.5 times as it was in Load Case 1) to make sure that there was not any damage on the right-wing side before Load Case 3 was performed.

3.2.1 Response of the deck slab – Load Case 2

The deflection of the girder segment at a number of critical points, as measured by installed dial indicators, is depicted in Fig. 21. It is evident from the load-displacement relationships that there is a linear behavior. At a load of 52 kN (40% of the maximum test load), noticeable changes occurred in the slopes of all the curves. This alteration occurred due to the formation of cracks at the construction joint as shown in Fig. 22. Following these changes in the deflection slopes, the

upper flanges continued to exhibit linear increments with the applied load (up to 120 % of calculated serviced load). The maximum deflection of the girder segment at the tip of the cantilever wing and the tip of the two right struts were 1.68 mm, 1.57 mm, and 1.60 mm, respectively. These deflection values were all quite similar, indicating that the slab maintained adequate stiffness during the test.

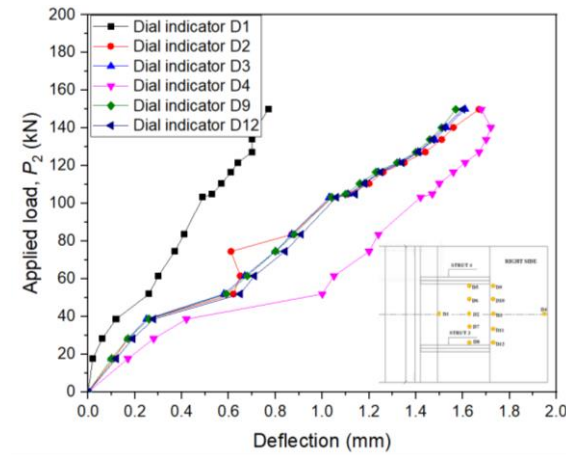


Fig. 21 – Load-deflection curves - Load Case 2



a) Cracks occurred at the construction joint at an applied load of 52 kN



b) Monitoring crack opening during the test

Fig. 22 – A crack occurred at the construction joint (right side of the wing slab)

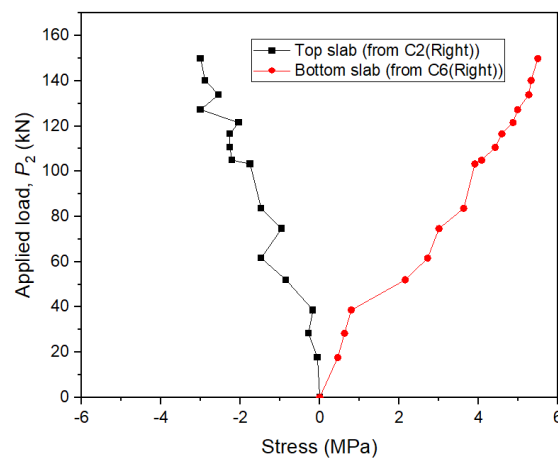


Fig. 23 – Stresses at top and bottom slab at construction joint (Section 3) - Load Case 2

The stresses at the top and bottom slabs at the construction joint, where significant positive moments were experienced, were calculated from the strain values obtained from attached strain gauges, as presented in Fig. 23. It is demonstrated that at the maximum applied load (120% of the calculated service load), the maximum compression and tensile stresses in the right flange were -3.00 MPa and 5.50 MPa, respectively. The test results indicate that while the maximum compressive stress was lower than the allowable compressive stress of -18 MPa (Table 3), the maximum tensile stress in the lower soffit exceeded the allowable tensile stress of 3.35 MPa. This is consistent with the occurrence of cracks at the construction joint location observed during the test, as evidenced by the measured deflection data. However, as shown in Fig. 23, at load of 52 kN where the crack began to initiate, tensile stress at the bottom of the joint was only approximately 1.11 MPa. This value is significantly smaller than the allowable tensile stress of 3.35 MPa. One possible explanation for this phenomenon is that the quality of the construction joint was not as good as designed, leading to early crack formation. Therefore, in real construction projects, special attention should be given to the construction joints to ensure their quality and integrity.

3.2.2 Response of the steel pipe strut - Load Case 2

The stress calculations obtained from strain measurements in Strut 3, both at its upper and lower ends, are presented in Fig. 24. Fig. 25 displays the corresponding axial force and moment values derived from these stress analyses. As depicted in Fig. 24, at the peak load, which equated to 120% of the calculated service load, the maximum tensile and compressive stresses in Strut 3 were recorded at 13.15 and -9.77 MPa, respectively. Notably, both values were observed to be below the allowable stress threshold for the strut, signifying an elastic response. Furthermore, Fig. 25 illustrates that under Load Case 2 scenario, Strut 3 not only withstood axial force but also bending moment. Remarkably, the bending moment is notably higher at the lower end of the strut compared to the upper end.

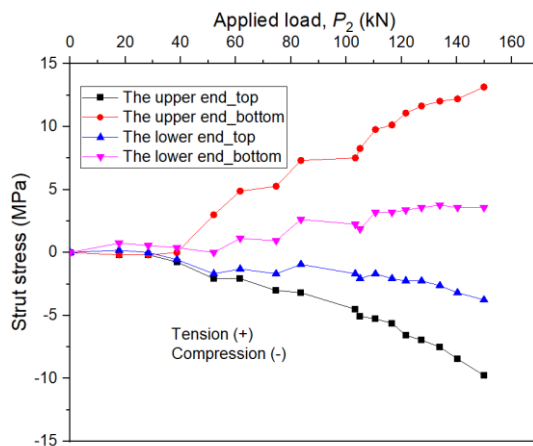


Fig. 24 - Load vs. developed stress in Strut 3 - Load Case 2

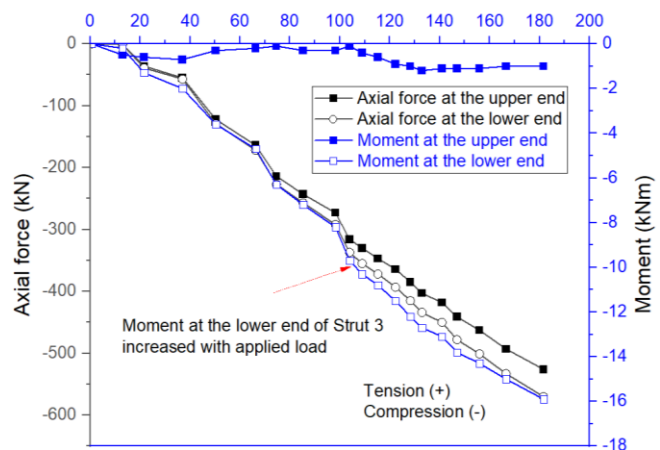


Fig. 25 - Load vs. developed internal forces in Strut 3 - Load Case 2

In general, under Load Case 2 scenario, where the applied load reached 120% of the calculated service load, both the slab and steel pipe struts demonstrated linear behavior. Deflection and concrete stresses at critical locations, as well as stress in the steel struts, remained within allowable limits. However, it is worth noting that the tensile stress value at the slab construction joint location exceeded the allowable threshold, leading to cracks at that specific region. This experimental observation emphasizes the importance of paying special attention to construction joints. Nonetheless, apart from this issue, the design proved to be structurally sound under Load Case 2 scenario.

3.3 Structural response under Load Case 3

Load Case 3 involved applying a concentrated force to the bridge deck precisely at the upper connection point of Strut 3 (Fig. 8(c)). The objective was to induce a maximum compression force within Strut 3 and scrutinize localized stress in the concrete at the junctions between Strut 3 and the concrete segment. The magnitude of the concentrated force was calculated to generate an equivalent stress in Strut 3 under the most unfavorable scenario at the Service Limit State, yielding a calculated value of P_3 at 270 kN. With the applied load of 270 kN, the total axial compressive load on Strut 3, inclusive of

the self-weight of the concrete deck, amounted to approximately 800 kN. Throughout the test, P_3 was incrementally increased up to 400 kN, representing 150% of the calculated value. At this level, the axial compression force exerted on Strut 3 peaked at approximately 930 kN corresponding to the design load at the Strength Limit State.

3.3.1 Response of the deck slab - Load Case 3

The deflection measurements at the slab cantilever tip, as well as at the tips of Struts 3 and 4, are presented in Fig. 26. It is evident that the relationship between the applied load and deflection followed a near-linear trend. Notably, there were abrupt changes in the deflection rate at an applied load of 231 kN, attributed to the emergence of cracks in the connection region (as depicted in Fig. 27). The measurements revealed that the width of these cracks was minimal, ranging from 0.02 to 0.04 mm, and remained stable throughout subsequent loading.

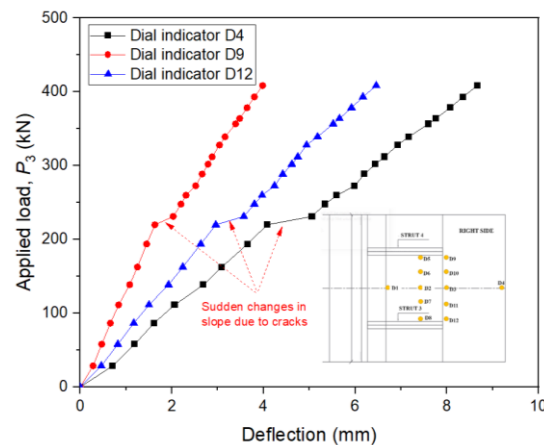


Fig. 26 – Load-deflection curves - Load Case 3



a) Cracks at the lower end of Strut 3



b) Monitoring cracks at the upper end of Strut 3

Fig. 27 – Cracks occurred in concrete under Load Case 3

3.3.2 Response of the steel pipe strut - Load Case 3

Results of stresses in Strut 3 (the strut below the applied load) at two cross sections of the upper and lower ends of Strut 3 are shown in Fig. 28. It is shown that Strut 3 behaved linearly with the applied load increased up to 400 kN, which corresponded to 150 % of the design load at Service Limit State and 100 % of the design load at Strength Limit State. The measured maximum compressive stress in Strut 3 was -62.8 and still much smaller compared to the yield stress of the steel pipe struts, showing a safety design.

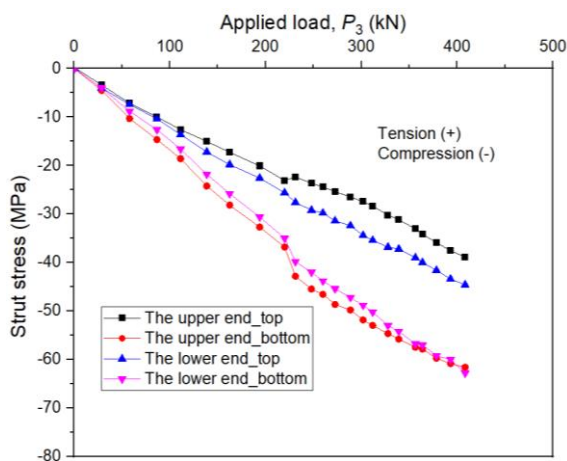


Fig. 28 - Load vs. developed stress in Strut 3 - Load Case 3

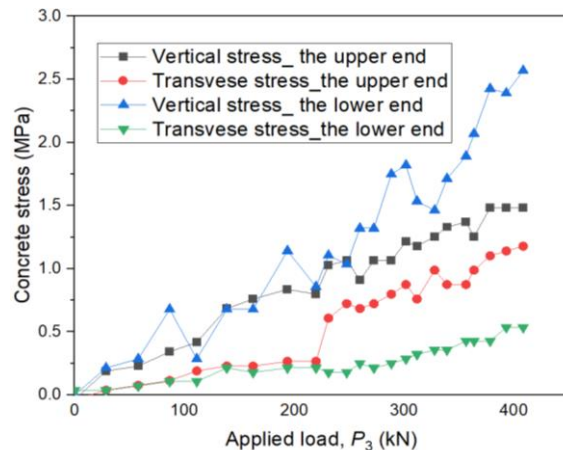


Fig. 29 – Concrete local stress at connections between concrete and Strut 3 - Load Case 3

Fig. 29 illustrates the local stresses in concrete at the two junctions between Struts 3 and the concrete of the girder segment. It is evident that the local stresses in both transverse and vertical directions at the connections gradually increase with the applied load. The maximum measured tensile stress in concrete was 2.45 MPa, which remains below the allowable tensile stress in concrete following TCVN 27205 Standard. Overall, under the Load Case 3 scenario, no abnormal signs in structural response were detected. When subjected to a load equal to 150% of the service load (100% of the load at Strength Limit State), Strut 3 remained within the elastic region, and both deflection and concrete stress at the most critical locations remained within acceptable limits. Thus, it can be concluded that the struts satisfied the load-bearing requirements at the maximum load under the Strength Limit State.

Experimental data showed that under the three load scenarios, except the cracks that occurred in the construction joint possibly due to connection quality, the girder segment behaved linear manner and had adequate load-bearing capacity to support the applied load equal to 150 % of the calculated load at Service Limit State. The steel struts ensured rigidity and load-bearing capacity under both Service Limit State and Strength Limit State according to 22TCVN 272-05 Standard. Welded joints and connections ensured quality during load testing. However, it should be noted that the girder segment was tested and simulated under static load with a load factor of 1.25 accounting for dynamic effect, and it may not reflect the actual dynamic effect of live load acting on the bridge. Thus, in future research, it is necessary to pay attention to the dynamic effects on the steel pipe struts, especially the connections at their two ends. In addition, construction joint quality is also needed to pay great attention to ensure the integrity of structures. To ensure adherence and prevent early cracks, a technique for treating the interface between concrete cast in different stages should be implemented.

4 Conclusions

A full-scale concrete box girder segment, incorporating diagonal steel pipe struts, was fabricated to assess the construction technology of this innovative bridge type. Three load cases, representing the most unfavorable conditions for the wing slabs and steel struts in the Service Limit State, were applied to the girder segment. The aim was to investigate the transverse behavior of strutted box girder bridges, with particular emphasis on the performance of strutted wing slabs, steel pipe struts, and connections under service loads. Based on observations during the fabrication process and test outcomes, the following conclusions are drawn:

- In concrete box girders utilizing steel struts, careful attention must be paid to the detailing of connections between the concrete and the pipe struts to ensure structural integrity and prevent cracks during fabrication. Heavy reinforcement arrangement and improper concrete casting methods in these connection areas can lead to reduced concrete strength and the formation of cracks in these critical zones. Additionally, precise technical instructions for tightening high-strength bolts in connections are essential to prevent cracks during construction.

- In Load Case 2, cracks were observed in the construction joint between the core segment and the cast-in-place wing slabs. Therefore, implementing a technique to manage the interface between concrete cast in different stages is necessary to prevent early cracks.
- Throughout all three load cases, with imposed loads ranging from 1.2 to 1.5 times the calculated load at the Service Limit State, both the wing slabs and the steel pipe struts maintained their load-bearing capacity and exhibited linear behavior. The steel struts effectively provided rigidity during load testing, indicating satisfaction with the design under service loads. However, it's important to note that the girder segment was tested under static loads, and therefore may not fully capture the dynamic effects of live loads acting on the bridge. Thus, special attention should be given to evaluating the dynamic effects to comprehensively assess the performance of the steel pipe struts under actual working conditions.
- The results obtained from this experimental program can offer valuable insights for professionals working on similar projects. Moreover, they can also serve as essential data for future numerical analyses, shedding light on various aspects of this innovative type of bridge.

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REFERENCES

- [1]- K. Saito, Yoda, T., Yamazaki, J., & Homma, A., Structural characteristics of box girder bridge with struttred wing slabs., in In Proceedings of the 11th fib International PhD Symposium in Civil Engineering, FIB 2016 (pp. 173-180), A.A. Balkema Publishers. (2016).
- [2]- A. Androus, H.M. Afefy, K. Sennah, Investigation of free vibration and ultimate behavior of composite twin-box girder bridges. *Journal of Constructional Steel Research*, 130 (2017) 177-192. doi:https://doi.org/10.1016/j.jcsr.2016.12.017.
- [3]- Y.C. Choi, B.H.J.J.o.B.E. Oh, Transverse modeling of concrete box-girder bridges for prediction of deck slab ultimate load capacity. 18(12) (2013) 1373-1382.
- [4]- P. Agarwal, P. Pal, P.J.I.J.f.E.M. Kumar Mehta, Box-girder bridges-modelling and analysis. 35(1 Regular Issue) (2022) 19-42.
- [5]- P. Agarwal, P. Pal, P.K. Mehta, Finite element analysis of reinforced concrete curved box-girder bridges. *Advances in Bridge Engineering*, 4(1) (2023) 1. doi:10.1186/s43251-023-00080-7.
- [6]- T. Gupta, M. Kumar, Flexural response of skew-curved concrete box-girder bridges. *Engineering Structures*, 163 (2018) 358-372. doi:https://doi.org/10.1016/j.engstruct.2018.02.063.
- [7]- J. Sali, R.P. Mohan, Parametric Study of Single Cell Box Girder Bridge under Different Radii of Curvature. *Applied Mechanics and Materials*, 857 (2017) 165-170. doi:10.4028/www.scientific.net/AMM.857.165.
- [8]- M. Zhuang, C. Miao, R. Chen, Fatigue Performance Analysis and Evaluation for Steel Box Girder Based on Structural Health Monitoring System. *Structural Durability \& Health Monitoring*, 14(1) (2020). doi:10.32604/sdhm.2020.07663.
- [9]- J. Cheng, H. Yao, Simplified method for predicting the deflections of composite box girders. *Engineering Structures*, 128 (2016) 256-264. doi:https://doi.org/10.1016/j.engstruct.2016.09.049.
- [10]- B.A. Eltaly, Y.B.I. Shaheen, B. Tayeh, A.A. Henish, Enhancing the behavior of reinforced concrete box girders under Two-point Loading: Numerical and experimental investigations. *Structures*, 52 (2023) 386-400. doi:https://doi.org/10.1016/j.istruc.2023.03.182.
- [11]- M. Zhou, J. Zhang, J. Zhong, Y. Zhao, Shear Stress Calculation and Distribution in Variable Cross Sections of Box Girders with Corrugated Steel Webs. *Journal of Structural Engineering*, 142(6) (2016) 04016022. doi:10.1061/(ASCE)ST.1943-541X.0001477.
- [12]- R. Gaspar, F.R. Stucchi, Web design of box girders concrete bridges. *Engineering Structures*, 57 (2013) 267-275. doi:https://doi.org/10.1016/j.engstruct.2013.09.012.
- [13]- B. Eltaly, M. El_Sayed, N. Meleka, K. Kandil, Torsion behavior of strengthened reinforced concrete box girders

- with openings: Analytical and experimental investigation. *Structures*, 60 (2024) 105908. doi:<https://doi.org/10.1016/j.istruc.2024.105908>.
- [14]- K.M. Sennah, J.B.J.J.o.B.E. Kennedy, Literature review in analysis of box-girder bridges. 7(2) (2002) 134-143.
- [15]- S.-T. Song, Y. Chai, S.E.J.J.o.b.e. Hida, Live-load distribution factors for concrete box-girder bridges. 8(5) (2003) 273-280.
- [16]- N. Hagiwara, K. Aoki, A. Kasuga, Y.J.J.o.p.c. Taira, Design and construction of the katsurashima viaduct–prestressed concrete corrugated steel web box girder bridge with ribs and struts. 47(3) (2005) 25-34.
- [17]- K.W.J.J.o.b.e. Shushkewich, Transverse analysis of strutted box girder bridges. 11(1) (2006) 33-47.
- [18]- K.W.J.P.j. Shushkewich, The strutted box widening method for prestressed concrete segmental bridges. 48(6) (2003) 64-81.
- [19]- J. Park, Y. Kim, I.J.C.S.E. Jang, Practical Behavior of PSC Box Girder Bridge with FRP Concrete Struts. 20(1) (2007) 71-77.
- [20]- J.-J. Song, Y.-K. Hwang, Y.-H. Lee, S.-H.J.J.o.t.K.i.f.s.m. Lee, inspection, An Experimental Study on the Application of FRP Tube to the Struts of PSC Box Girder Bridge. 13(1) (2009) 179-185.
- [21]- J.-H. Park, Park, C-S., Jang, I-H., & Lee, H-W., Development of New PSC Box Girder Bridge with Concrete-filled FRP Struts, in International Commemorative Symposium for the Incheon Bridge, Korea, 23 Sep., 2009. (2009).
- [22]- K. Saito, A. Homma, T. Sato. A Time-dependent Analysis Applied to the Superstructure Design of the Uchimaki Viaduct. in Proceedings of fib'06 - The 2nd International Congress, June 5-8, 2006. Naples, Italy. (2006).
- [23]- K. Saito, The Superstructure Design of the Uchimaki Viaduct. 第 11 回 PC シンポジウム, 2001 年 11 月, (2001).
- [24]- AASHTO, AASHTO LRFD bridge design specifications, American Association of State Highway and Transportation Officials, [2014]. (2017).
- [25]- K. Osada, T. Kanamoto, K. Saito, T. Arai, Progressive Erection Applied to Box Girder with Strutted Wing Slab.
- [26]- 22TCN:272-05, "Specification for bridge design", Ministry of Transport of Vietnam. (2005).
- [27]- AASHTO LRFD bridge design specifications 2nd edition. American Association of State Highway and Transportation Officials, [1998], 1998.