

Monitoring of Crack Flank Movement in a Steel Orthotropic Deck

Rudolf ÁROCH^{1,*}, Tomáš KLAS¹, Ján BRODNIANSKY¹

¹ Department of Steel and Timber Structures, Faculty of Civil Engineering, Slovak University of Technology, Radlinského 11, Bratislava, Slovakia.

* corresponding author: rudolf.aroch@stuba.sk

Date of Submission: 15 August 2025
Revision Date: 30 September 2025
Date of Acceptance: 30 September 2025



Civil and Environmental Engineering
Journal of the Faculty of Civil Engineering | University of Žilina

Abstract

Fatigue cracking is a common issue in steel orthotropic bridge decks subjected to long-term dynamic loading. When repairs are required, especially by welding, it is typically necessary to close the bridge to traffic to ensure proper weld quality. However, bridge closures can lead to significant societal and economic impacts, including traffic congestion, long detours, increased emissions, and loss of time. Therefore, the ability to carry out repair welding without full traffic closure is of considerable interest for infrastructure operators and public authorities.

The first essential step in enabling such in-service repair is to assess the dynamic behaviour of existing fatigue cracks under traffic loads. Specifically, it is crucial to monitor the crack flank movements to determine whether they remain within acceptable limits during ongoing traffic, thereby permitting welding without compromising the repair quality.

This paper presents the development and deployment of a specialized monitoring system designed to measure crack flank displacements in real time. Crack flank movement amplitudes and frequency responses were recorded and analysed. Measured data were compared with results from similar monitoring campaigns conducted in other countries.

Keywords

Orthotropic decks of steel bridges; Repair welding; Crack flank movement; Structural health monitoring; Service traffic loading.

1. Introduction

Many bridges built in the late 1960s and early 1970s were designed according to the engineering principles, construction technologies, and materials available at that time. Traffic volumes and axle loads were also substantially lower than those experienced today. Over the decades, these factors have contributed to the emergence of various forms of damage, including fatigue cracking in the details of orthotropic steel bridge decks.

Conventional repair practices involve removing the damaged welds and replacing them with new ones, often accompanied by a change in weld type (for example, replacing fillet welds with butt welds).

When traffic moves across a bridge containing cracks, relative displacement occurs between the crack flanks. To ensure the production of high-quality welds, such repairs are generally carried out under complete traffic closure. Determining the conditions under which welds of the required quality can be executed while allowing traffic to remain –

whether through lane restrictions, diversion to adjacent lanes, or speed reduction — holds significant societal and economic value.

2. State of the Art

The first systematic investigations into welding under cyclic loading conditions were carried out in Japan during the 1980s. Horikawa et al. (1983) examined the behaviour of welded joints subject to both cyclic and static loads through a series of tests. Their experiments used slot-shaped specimens joined along the edges. Due to the specific specimen geometry and the use of cylinder stroke control – rather than gap opening – to regulate the load amplitude, no cracking was observed for any tested combination of frequency and amplitude.

The primary challenge in welding with moving flanks is the formation of hot cracks in the initial weld layer. When weld flanks undergo dynamic motion during welding, additional deformation and strain are introduced. These, combined with shrinkage stresses from cooling, significantly increase the risk of hot crack formation.

In his doctoral thesis, Wichers (2006) employed a dedicated test setup in which the plates to be welded were mounted on sliding supports, with gap opening controlled by sensors. He demonstrated that weld seam integrity under oscillatory loading is primarily governed by the interaction between movement amplitude and frequency. The seam flank displacements investigated varied from 0.1 mm to 0.8 mm, and the frequencies from 0.1 Hz to 7 Hz. Hot cracks occurred in all tested amplitude-frequency combinations, typically initiating at the weld root. The propagation pattern and crack width were found to be strongly influenced by both parameters.

Peil & Wichers (2005) and Peil et al. (2008) extended the investigation to a broader range of movement amplitudes. Their welding tests confirmed that both the number and size of cracks are highly dependent on the magnitude of the displacement amplitude and the frequency.

To address the feasibility of welding under service loading conditions, the FOSTA research project reported by Müller et al. (2023) focused on determining the maximum allowable relative movement of weld flanks that still permits defect-free welding. Partial findings have also been published by Begemann et al. (2023, 2004, 2005) and by Unglaub et al. (2024). The study began by identifying a damaged bridge detail and instrumenting it to record local deformations under typical traffic loading. A purpose-built measurement system monitored the relative displacement of crack flanks, and the resulting time-histories were analysed to quantify displacement amplitudes and frequencies induced by traffic. Subsequently, a large-scale experimental setup was developed to impose controlled relative displacements between the weld flanks of two plates before joining. Welding trials were conducted under various opening amplitudes and frequencies using a displacement-controlled test rig. It was found that sound joints could still be produced under a maximum relative movement of 0.1 mm at a frequency of 2 Hz. A repair method enabling defect-free butt joints accessible from both sides was implemented, and the resulting fatigue specimens were evaluated for weld quality in accordance with ISO 5817 before being subjected to fatigue testing. The fatigue performance of welds produced under service loading was found to be comparable to that of specimens welded under no-load conditions. An important observation was that hot cracks occur under service loading even at small gap opening movements.

Further research is ongoing within the project *“Repair Welding of Fatigue-Loaded Steel Structures in Service with Complex Weld Details and Limited Accessibility”* (FOSTA Research Project P1742 / IGF No. 23115 BG).

3. The SNP Bridge

3.1. Description of the Structure

The SNP Bridge, spanning the River Danube in Bratislava, the capital of the Slovak Republic, is the country's largest-span bridge. It is a steel cable-stayed structure with beam spans measuring 74.80 m + 303.00 m + 54.00 m. The

bridge beam comprises a steel, closed, double-box cross-section. The stay cables, arranged in the bridge's central plane, adopt a fan-shaped configuration. The structure is asymmetrical, supported by a single A-shaped steel pylon inclined longitudinally, with its apex leaning away from the river. At the top, the pylon houses a distinctive restaurant and observation deck.

The orthotropic deck features a steel deck plate with a minimum thickness of 12 mm, stiffened by longitudinal trough-shaped trapezoidal stiffeners. The stiffeners, cold-formed from steel plate with a minimum thickness of 6 mm, are 300 mm in height and spaced at 600 mm centres. Crossbeams are spaced at 3000 mm and designed as continuous beams with two spans of 6300 mm and two cantilevered ends of 4200 mm. At the cantilever tips, they are interconnected by longitudinal edge girders (Figure 1).

The bridge was officially opened to traffic in 1972.

3.2. Types and Causes of Cracks

In 2019, several cracks were identified in the welds of the orthotropic bridge deck. Diagnostic investigations determined that these were predominantly fatigue cracks, initiating at the weld root and propagating through the weld cross-section. The cracks are concentrated along the wheel paths in the outer traffic lanes, which primarily carry urban bus and passenger car traffic; heavy goods vehicles are prohibited.

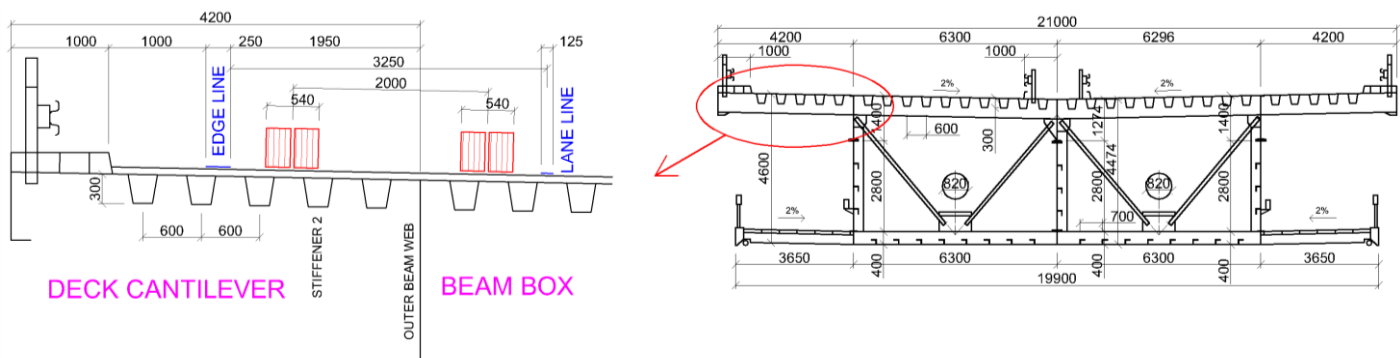


Figure 1: Detail of the bridge cross-section and position of vehicles in the outer traffic lane

Stiffener-to-deck plate weld (Figure 2)

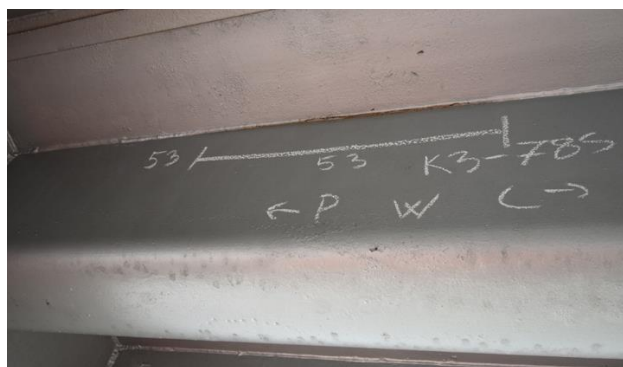


Figure 2: Crack in the weld between the longitudinal stiffener and the deck plate

The longitudinal stiffener is connected to the deck plate by single-sided fillet welds with a leg length of 5 mm. From today's standpoint, this weld is undersized. The stiffener plate edge is not parallel to the deck plate, and the gap between the two plates frequently exceeds 0.5 mm. The fillet weld penetration is only up to 2 mm, producing a pronouncedly

asymmetrical profile. Cracks develop beneath the wheel paths of heavy vehicles (in stiffener 2, see **Chyba! Nenašiel sa žiaden zdroj odkazov.**) and appear at random locations along the stiffener span.

Stiffener-to-crossbeam weld (Figure 3)

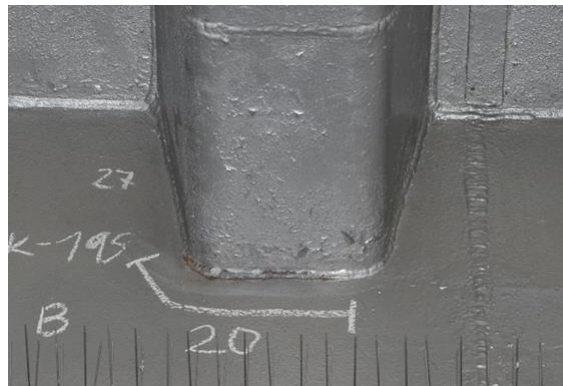


Figure 3: Crack in the weld between the longitudinal stiffener and the crossbeam

Throughout the bridge, longitudinal stiffeners are discontinuous at the crossbeam locations, where the crossbeam web interrupts the stiffeners. The stiffeners are welded directly to the crossbeam web using half-V butt welds reinforced by fillet welds. Significant misalignment between stiffeners on opposite sides of the crossbeam web is common, in some cases reaching several centimetres – well beyond the ± 2 mm tolerance required by current standards. Cracks typically initiate at the rounded ends of the stiffeners and propagate through either the flange or web welds.

Stiffener splice weld (Figure 4)



Figure 4: Crack in the weld at the longitudinal stiffener splice

The longitudinal stiffener splice is formed with a butt weld on a permanent root backing strip. Its location is suboptimal, placed at midspan – where bending moments are greatest – rather than at a point of zero moment. Misalignment frequently exceeds 1.0 mm. The specified gap between weld preparation edges, 3.0 mm, is significantly below the recommended range of 8-12 mm (FprCEN/TS 1993-1-901:2025). When the gap is less than 4 mm, the detail category decreases from 100 to 36 (FprEN 1993-1-9:2024). In addition, the tack welds of the backing strip are intermittent fillet welds placed outside the butt weld profile rather than within the butt joint itself (FprCEN/TS 1993-1-901:2025).

The primary factors contributing to crack formation are as follows:

- Increased loading over the bridge's service life.

- Design deficiencies – particularly the small deck plate thickness – reflecting the absence of modern code provisions at the time of construction.
- Excessive manufacturing tolerances.
- Poor workmanship, including undersized or asymmetrical welds, continuous and intermittent undercuts, notches between weld beads, irregular weld surfaces, spatter, dents in fillet welds, insufficient penetration, and other deficiencies.

The use of steel with inadequate properties can be excluded as a cause. The orthotropic deck is fabricated from steel grade 11 523.1. Mechanical tests on plate specimens cut from the structure confirmed compliance with both the original ČSN 41 1532 standard and the current EN 10025-2 requirements for hot-rolled flat products of steel grade S355J2. Impurity levels, including sulphur and phosphorus content, are low for steel produced in the late 1960s and early 1970s, and the steel is exceptionally well killed. Chemical analysis revealed no factors likely to impair weldability or service performance. Ultrasonic testing further confirmed that the plates are sound through their thickness and free from significant delamination.

4. Methodology

4.1. Research Design

The study focused on three typical types of fatigue cracks identified on the SNP Bridge. For each crack type, relative crack flank displacements under traffic loading were monitored and later compared with findings from recent literature.

4.2. Materials and Procedures

Measurements were carried out using an HBM Spider8 data acquisition system connected to an external battery. Displacement was recorded using HBM WA50 inductive standard displacement transducers. These sensors, equipped with spring-loaded plungers, were mounted to the structure using custom 3D-printed fixtures and magnetic bases.

Traffic on the bridge was simultaneously recorded using a video camera to correlate measured displacements with specific loading scenarios. The measurement sampling frequency was 400 Hz.

4.3. Data Collection

Crack flank displacement was measured for the following cases:

- Stiffener-to-deck plate weld.
- Stiffener-to-crossbeam weld.
- Stiffener splice weld.

The greatest crack movement was anticipated when a bus travelled in the outer lane. Additional scenarios were also examined, including:

- Passenger car in the outer lane.
- Bus in the inner lane.
- Passenger car in the inner lane.

5. Measurement Results

Representative measurement cases are presented below.

5.1. Case 1 – Stiffener-to-Deck Plate Weld

- Crack location: Vienna walkway, Section 27, Stiffener 2, Vienna side, approx. quarter-span of stiffener.
- Crack length: ~76 cm.
- Sensors: Vertical and horizontal LVDTs (Figure 5).



Figure 5: LVDT placement at Section 27 - vertical and horizontal

Traffic Scenario 1: Regio bus + Car 1 + Car 2 + Car 3 in the outer lane

Measured crack flank displacements for this scenario are shown in Figure 6, with values listed in Table 1.

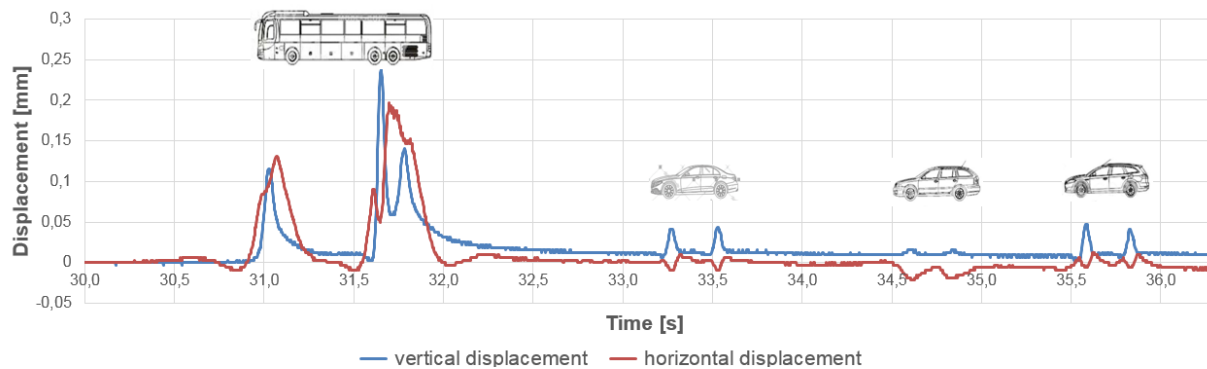
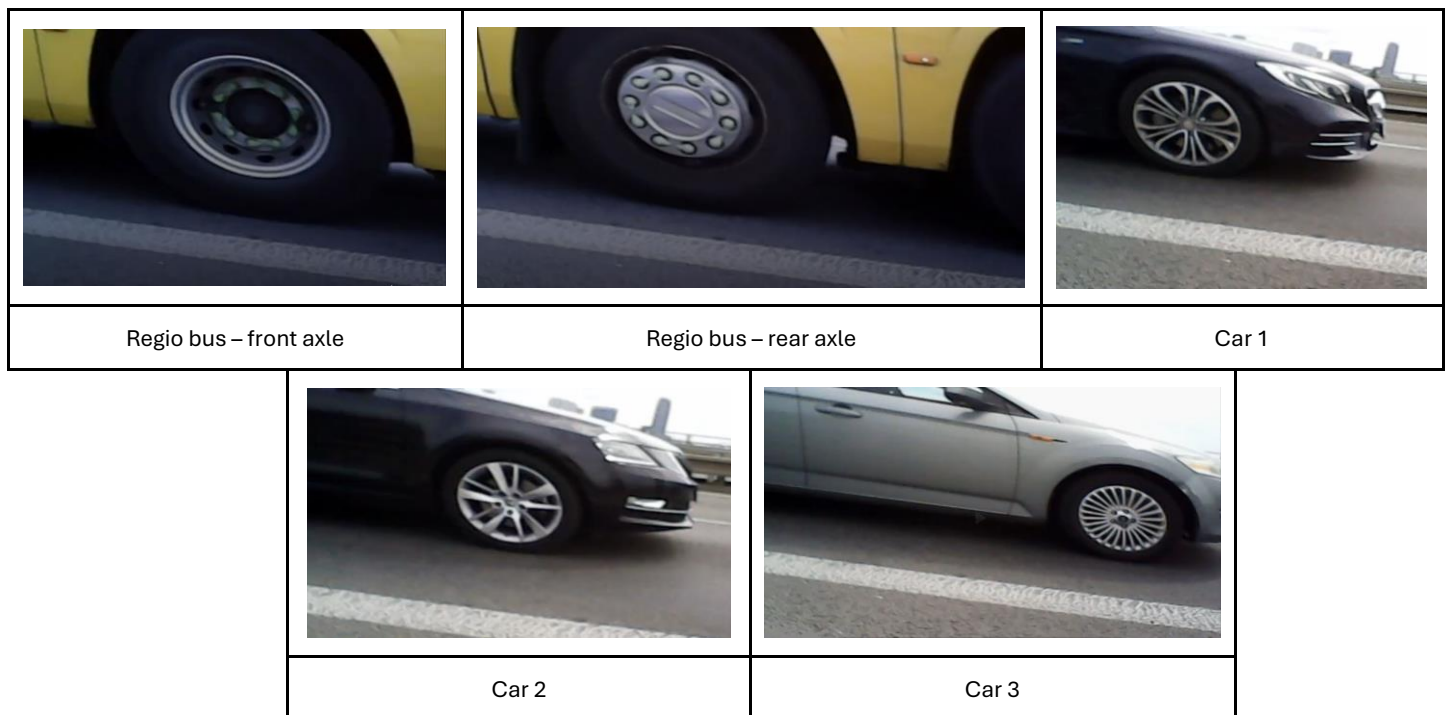


Figure 6: Measured displacement of crack flanks - Case 1, Scenario 1

Table 1: Vertical and horizontal displacements - Case 1, Scenario 1

Load	Car axle	Displacement [mm]	
		Vertical	Horizontal
Bus Regio	Front	0.116	0.1131
	Rear 1	0.2375	0.194
	Rear 2	0.141	0.153
Car 1	Front	0.041	-0.009
	Rear	0.044	-0.009
Car 2	Front	0.006	-0.022
	Rear	0.006	-0.019
Car 3	Front	0.047	0.013
	Rear	0.038	0.013

Vehicle positions during the measurement were determined from video stills (Figure 7). The edge line width of 25 cm was used to estimate the precise wheel position relative to the underlying deck stiffeners.

**Figure 7:** Vehicle positions relative to the edge line – Case 1**Traffic Scenario 2:** Bus SOR in the outer lane

Measured crack flank displacements for this scenario are shown in Figure 8, with values listed in Table 2.

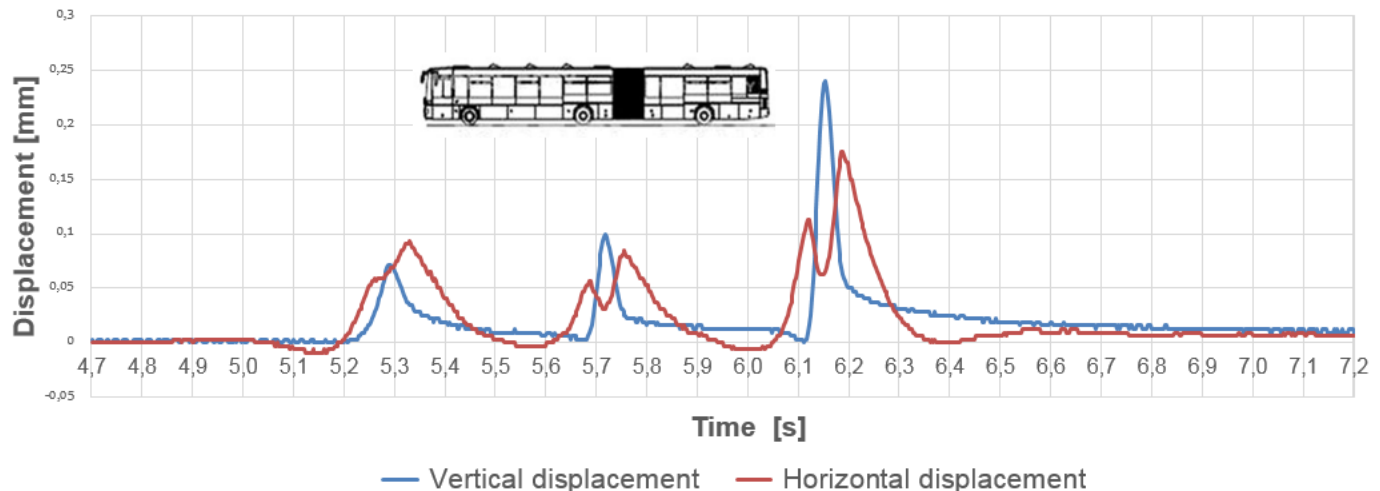


Figure 8: Measured displacement of crack flanks - Case 1, Scenario 2

Table 2: Vertical and horizontal displacements - Case 1, Scenario 2

Load	Car Axle	Displacement [mm]	
		Vertical	Horizontal
Bus SOR 1	Front	0.072	0.094
	Middle	0.097	0.084
	Rear	0.241	0.172

The maximum vertical relative movement was 0.241 mm (bus) versus 0.047 mm (cars), indicating that passenger cars generated approximately five times less vertical movement. The maximum horizontal relative movement was 0.194 mm (bus) versus 0.022 mm (cars), with cars producing about nine times less horizontal displacement. Traffic in the inner lane had negligible effect on this crack, located in the cantilever portion of the orthotropic deck.

In cases where cracks extended over the full stiffener span (~3 m), the highest measured displacements were unexpectedly located at the quarter span rather than at midspan, in some cases up to ten times greater than expected. This suggests partial restraint of the crack flanks, which is likely to be released during repair grinding, potentially increasing crack flank movement significantly.

5.2. Case 2 – Stiffener-to-Crossbeam Weld

- Crack location: Vienna walkway, Section 27, Stiffener 3, Bratislava side, at stiffener bottom flange.
- Crack length: ~19 cm.

Traffic Scenario 3: Bus SOR in the outer lane

Measured displacements are shown in Figure 9 and Table 3.

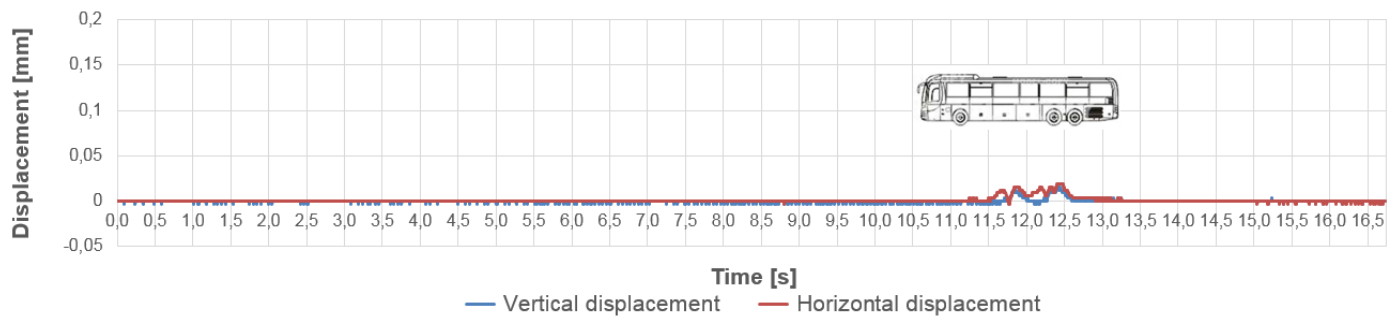


Figure 9: Measured displacement of crack flanks - Case 2, Scenario 3

Table 3: Vertical and horizontal displacements - Case 2, Scenario 3

Load	Car axle	Displacement [mm]	
		Vertical	Horizontal
Bus Regio	Front	0.013	0.016
	Rear 1	0.013	0.016
	Rear 2	0.016	0.019

The maximum measured relative movement (0.016 mm vertical, 0.019 mm horizontal) was approximately one-tenth of that observed in Case 1. This lower displacement is attributed to the shorter crack length (19 cm). Passenger cars produced negligible displacement, and inner-lane traffic had no significant influence on this cantilever-located crack.

5.3. Case 3 – Stiffener Splice Weld

- Crack location: Budapest walkway, Section 84, Stiffener 3, stiffener bottom flange at mid-span.
- Crack length: ~20 cm.
- Sensor: Horizontal LVDT (Figure 5).

Traffic Scenario 4: Car 1 + Car 2 + Bus SOR + Car 3 in the outer lane



Figure 10: LVDT placement at Section 84 - horizontal

Measured displacements are shown in Figure 11 and Table 4.

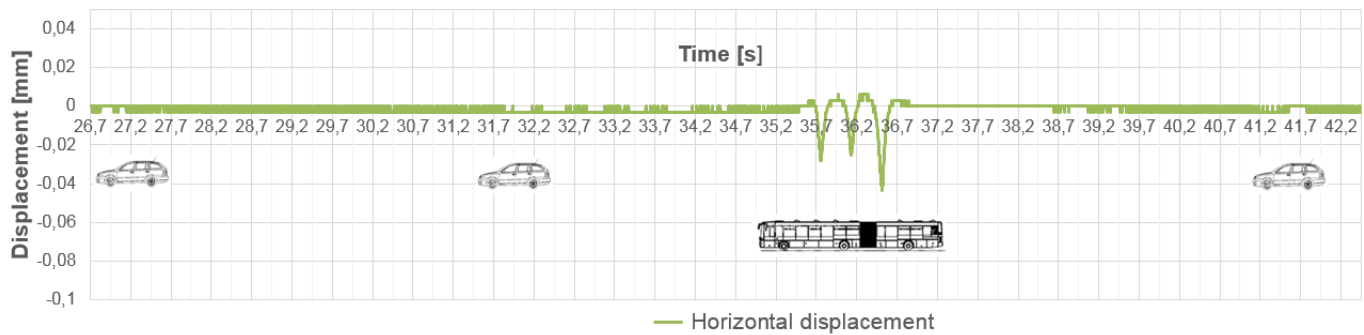


Figure 11: Measured displacement of crack flanks - Case 3, Scenario 4

Table 4: Horizontal displacements - Case 3, Scenario 4

Load	Car axle	Horizontal displacement [mm]
Car 1	Front	0
	Rear	0
Car 2	Front	0
	Rear	0
Bus SOR	Front	-0.028
	Middle	-0.025
	Rear	-0.044
Car 3	Front	0
	Rear	0

The maximum horizontal relative displacement was 0.044 mm (bus), with passenger cars producing negligible movement. Inner-lane traffic again had no measurable influence on this crack, located in the cantilever portion of the deck.

6. Discussion

6.1. Interpretation of Results

The measurements demonstrate that the largest relative crack flank movements occur under heavy traffic loading, in this case from buses, while passenger cars have a negligible effect. Traffic in the adjacent (inner) lanes does not significantly influence crack flank movement, consistent with the findings reported in DVS 1709 (2022).

The maximum recorded relative displacements were:

- 0.241 mm - stiffener-to deck plate crack.
- 0.019 mm - stiffener-to-crossbeam crack.
- 0.044 mm - stiffener splice crack.

When compared with measurements on the Duisburg-Neuenkamp Rhine Bridge in Germany – where maximum movements were 0.1 mm at 2 Hz (Müller et al., 2023) – the SNP Bridge stiffener-to-deck plate crack values are substantially higher. According to the conclusions of Müller et al. (2023), relative movements above 0.1 mm prevent the execution of a sound repair weld. Consequently, traffic directly above the repaired location must be excluded during welding operations.

Although the SNP Bridge is closed to heavy goods vehicles over 21 tonnes, city buses and some intercity buses are permitted. The frequency of such loading events is relatively low. However, a more detailed axle-by-axle analysis indicates that higher frequency ranges can still occur during these events.

6.2. Limitations

The study represents a preliminary investigation with a limited number of measurement cases and traffic scenarios. The results are therefore specific to the observed conditions on the measurement date and may not fully capture seasonal, temperature-related, or long-term variations. Additionally, the monitoring period was short, and more extended data collection could improve the statistical reliability of the results. Finally, the findings are limited to cracks located in the cantilever portions of the orthotropic deck; results may differ for cracks in other regions.

7. Conclusion

This study presents the first set of preliminary measurements of relative crack flank movements on the SNP Bridge. The results show that, particularly for stiffener-to-deck plate cracks, movements during bus passages exceed the acceptable threshold for executing sound repair welds. This indicates that traffic should, at minimum, be diverted away from the lane undergoing repair.

Recent research has demonstrated that hot cracks can form even at small relative movements. At an initial displacement amplitude of 0.1 mm and a frequency of 2 Hz, no surface cracks were observed during or after welding; however, detailed inspection revealed minor hot cracks in the root pass — an inherent feature of welding under service loading (Müller, et al., 2023). In those experiments, sound welds could still be achieved by grinding out the defective root, a solution only feasible when both sides of the joint are accessible. In the case of the SNP Bridge cracks, this approach is not possible, and alternative procedures must be developed.

Hot cracks formed during repair welding under cyclic loads are primarily solidification cracks. Welding parameters influence dendrite growth direction; orienting dendrites parallel to the welding direction – achievable at lower welding speeds – can improve resistance to solidification cracking. However, this conflicts with the need for higher welding speeds to minimise heat damage to the adhesive and waterproofing layers of the wearing surface (Ároch, 2022). Additional factors affecting hot crack initiation include alloy composition and the restraint imposed by the surrounding bridge structure (Begemann, 2024). These influences require further investigation.

During repair welding, relative displacement of crack flanges must be prevented by:

- Using temporary plate fixations that allow sectional or partial repair.
- Scheduling welding during periods of minimal crack-gap movement (e.g. night hours or weekends).
- Temporarily closing or diverting traffic from the affected lane.
- Reducing vehicle speeds over the repair site.

Further research into residual stress formation during repair welding is also recommended. Repair welding of fatigue cracks under service conditions remains a complex challenge that requires continued study to develop safe, effective, and durable solutions.

Acknowledgements

This paper has been supported by the Scientific Grant Agency (VEGA) – Grant from the research programme No. 1/0654/24.

Author Contributions

R.A. designed the study and supervised the project. **T.K.** conducted the experiments and contributed to manuscript writing. **J.B.** conducted the experiments and performed the data analysis. All authors critically reviewed and approved the final version of the manuscript and agreed to be accountable for all aspects of the work.

Disclosure of Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data Availability Statement

The data supporting the findings of this study are available from the corresponding author upon reasonable request.

References

- Ároch, R. et al. (2019). SNP Bridge – Extraordinary Inspection, Including Viaducts, Performance Analysis, and Assessment of Actual Structural and Technical Condition (in Slovak). Dept. of Steel and Timber Structures, Slovak University of Technology in Bratislava. Commissioned by the Department of Road Management. Capital City of the Slovak Republic Bratislava, December 2019.
- Ároch, R. et al. (2021). SNP Bridge Deck – Diagnostics, Structural Analysis, Condition Assessment, and Rehabilitation Design (in Slovak). Dept. of Steel and Timber Structures, Slovak University of Technology in Bratislava. Commissioned by the Department of Road Management, Capital City of the Slovak Republic Bratislava, December 2021.
- Ároch, R., Klas, T., & Petřík, A. (2022, May). *Temperatures during Weld Repair on Orthotropic Steel Decks*. IABSE Symposium Prague 2022 – Challenges for Existing and Oncoming Structures, Prague, Czech Republic.
- Begemann, F., Unglaub, J., & Thiele, K. (2023). Welding under service conditions - Influence of different boundary conditions during welding. *ce/papers* 6 (2023), No. 3-4 2601-2605. <https://doi.org/10.1002/cepa.2667>
- Begemann, F., Müller, J., Unglaub, J., Dilger, K., Thiele, K., & Hensel, J. (2024). Welding under service conditions—Resulting weld quality and fatigue strength assessment. *Engineering Failure Analysis*, 160, 108142. <https://doi.org/10.1016/j.engfailanal.2024.108142>
- Begemann, F., Unglaub, J., Hensel, J., & Thiele, K. (2025). Reparaturschweißen an Brücken im Betrieb. *Stahlbau* 94, H. 2, 62–69. <https://doi.org/10.1002/stab.202400051>
- FprEN 1993-1-9:2024 Eurocode 3. Design of Steel Structures. Part 1.9: Fatigue. CEN/TC 250, 2024.
- FprCEN/TS 1993-1-901:2025 Eurocode 3. Design of Steel Structures. Part 1-901: Fatigue Design of Orthotropic Bridge Decks with the Hot Spot Stress Method. CEN/TC 250, 2025.
- Horikawa, K., Suzuki, H., & Imi, K. (1983). Repair Welding on Bridges in Service Condition (Welding Mechanics, Strength & Design). *Transactions of JWRI*. 12(2), 309-315. <https://doi.org/10.18910/7849>
- Peil, U., & Wichers, M. (2005). Schweißen unter Betriebsbeanspruchung Experimentelle Untersuchungen unter einachsiger Nahtflankenbewegung während des Schweißprozesses. *Stahlbau* 74, H. 12, 891-900. <https://doi.org/10.1002/stab.200590217>
- Peil, U., Hoeft, T., Reininghaus, M., & Wichers, M. (2008). Schweißen unter dynamischer Einwirkung. *Stahlbau* 77, H. 10, 721-727. <https://doi.org/10.1002/stab.200810086>
- Merkblatt DVS 1709 (2022). Instandsetzung und Verstärkung von geschweißten Stahlbrücken. DVS – Deutscher Verband für Schweißen und verwandte Verfahren E.V., November 2022.
- Müller, J., Tóth, T., Dilger, K., Begemann, F., Unglaub, J., & Thiele, K. (2023). Schweißen unter Betriebsbeanspruchung unter Einhaltung normgerechter Anforderungen an Nahtgüte und Schwingfestigkeit, FOSTA – Forschungsvereinigung Stahlanwendung e. V., 01.04.2019 - 30.09.2022, Forschung für die Praxis P 1362, IGF-Nr.: 20678 N, ISBN 978-3-96780-156-9.

- Unglaub, J., Begemann, F., Hensel, J., & Thiele, K. (2024). Welding under service conditions – Monitoring, repair strategy and rehabilitation. SMAR 2024 – 7th International Conference on Smart Monitoring, Assessment and Rehabilitation of Civil Structures. *Procedia Structural Integrity* 64 (2024) 918–924.
- Wichers, M. (2006). Schweißen unter einachsiger, zyklischer Beanspruchung. Experimentelle und numerische Untersuchungen. PhD. Thesis. Fakultät Architektur, Bauingenieurwesen und Umweltwissenschaften der Technischen Universität Carolo-Wilhelmina zu Braunschweig.

How to Cite This Article

Ároch, R., Klas, T., & Brodniansky, J. (2026). Monitoring of Crack Flank Movement in a Steel Orthotropic Deck. *Civil and Environmental Engineering*, Vol. 22, Issue 1, 448-460. <https://doi.org/10.2478/cee-2026-0048>
