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Case study on emergency measures for maintenance and re-support of an embankment supporting a bridge abutment

Sarmad Al-Tameemi, Oliver Bernecker
GrundWerk GmbH & Co. KG, Stuttgart, Germany, al-tameemi@gw-gi.de

ABSTRACT: Many bridges are subjected to a range of damages over time, and among these, one of the most critical and potentially hazardous issues is the failure of slope stability around the bridge abutment, which can significantly compromise the overall structural integrity and safety of the bridge. Therefore, it is crucial to prevent damage to the foundation by conducting routine inspections and monitoring of the abutments as well as the surrounding supporting soil. A case study was conducted for an emergency measure for maintenance and re-supporting a bridge abutment located in southern Germany, which was experiencing slope stability failure due to partial failure in the anchorage system that supports the slope. It was determined that additional support for the slope was necessary, which involved the installation of additional ground anchors with an adjustable reaction support designed to replace the aging anchor system and re-balance the load distribution in the system that was on the verge of complete failure. Furthermore, to reinforce the embankment surface, a steel grid was installed along with soil nails, providing enhanced stability and long-term protection against potential slope movements. By including a long-term monitoring system, these measures present a flexible and adaptable mechanism for the long-term control and monitoring of similar emergency situations.

KEYWORDS: Slope stability, anchors, soil nails, monitoring, bridge abutment.

1 INTRODUCTION

The bridge, constructed in 1980, serves as a connection between two sides of a landscaped area spanning a highway in southern Germany. The bridge length is about 71 m, which is supported by two abutments and two piers, with both abutments founded on sloped terrain. Figure 1 shows a view toward the north.

Figure 1. General view of the bridge toward the north abutment.

This study focused on the northern embankment, which was supported by two ground anchors (A1 and A2) and later with additional anchor A3; these are accompanied by pressure load gauges (PG) for continuous monitoring of the anchor load over the lifespan. The embankment has an overall height of approximately 12.0 m and about 18.0 m width, with an average inclination of 33°. In the area adjacent to the bridge abutment, the embankment extends over a length of roughly 12.0 m. The abutment footing is embedded at 7.5 m depth below the highest point of the embankment surface, while the anchor heads are installed at 1.5 m beneath the base of the footing.

The subsoil profile consists of a 4.0 m thick layer of local loam, underlain by approximately 4.4 m of weathered claystone, followed by a layer of hard limestone. The groundwater table was observed at a maximum level of approximately 9.0 m below the highest ground surface. To relieve hydrostatic pressure, a drainage pipe was installed above the level of the anchors. Figure 2 presents a side view for the embankment with the existing anchorage system.

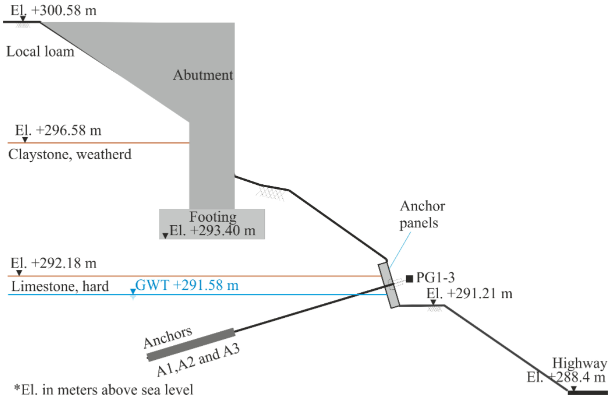

Figure 2. Side view of the embankment and abutment with the existing anchorage system.

In 2024, noticeable slope movements and service damages were observed in the area adjacent to the northern abutment, raising concerns about the safety of the slope and the bridge foundation. At the interface between the anchor panels and the overlying cobblestone layer, deformations of up to 10 cm were recorded. Significant cracking was observed in the slope surface with maximum crack depths reaching 40 cm and up to 5 cm widths. In addition, the stairway exhibited noticeable horizontal displacement, which caused it to become partially non-functional. Numerous cobblestones became dislodged and fell as a consequence of ongoing slope deformation. Figure 3 and Figure 4 show the deformations and damages in the embankment and stairway.

On the other hand, the abutment exhibited a noticeable rotation, as it can be noticed that the joint on the pavement between the bridge and the abutment was smaller on the west side and bigger on the east side of the pavement. However, the bridge and piers show some insignificant signs of cracks and remain functional.

Emergency stabilization measures were therefore required to prevent further displacement and structural compromise. This study focuses on the observed failure mechanism, the implemented remediation concept, and the monitoring strategy applied at the north abutment.

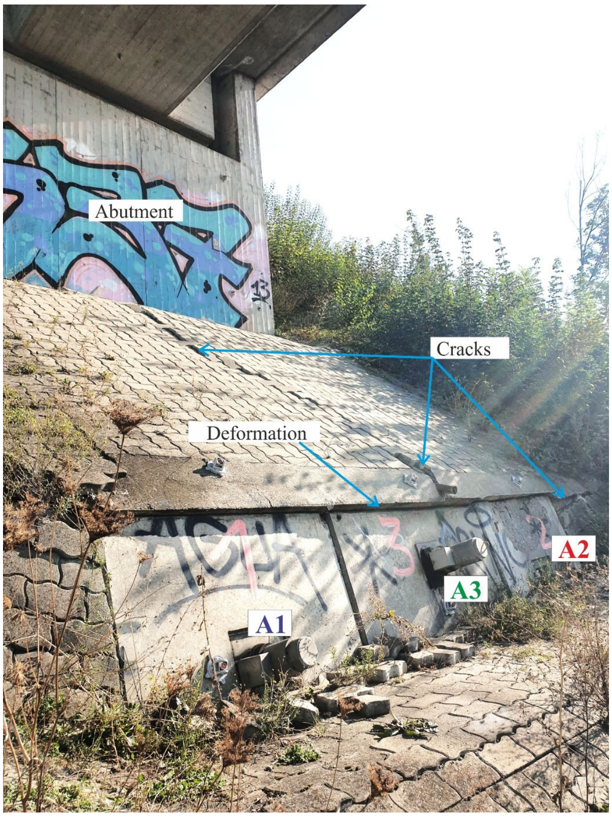

Figure 3. Cracks and deformation in the cobblestone.

Figure 4. Deformation of the steps and crack measures.

2 FAILURE MECHANISMS

Anchored system is a well-established method for stabilizing slopes and supporting geotechnical structures, particularly in transport infrastructure. Their effectiveness depends on appropriate design, installation, and long-term monitoring to ensure structural performance under varying ground conditions.

The design and execution of ground anchors in Germany are regulated by DIN EN 1537 (2014), which defines methods and procedures for load testing, bond lengths, and corrosion protection for both temporary and permanent anchors. These specifications are further elaborated in national guidelines such as the recommendations EAB (2021), which provide detailed requirements for acceptance criteria, proof loading, and safety margins specific to German practice.

The anchors were originally designed to carry a load of 400 kN. However, subsequent investigations revealed that anchor A1 could only sustain a maximum load of 175 kN, whereas anchor A2 retained its full load-carrying capacity ≥ 400 kN. As a precautionary measure and to maintain overall slope stability, both anchors were stressed to a lock-off load of 150 kN.

The anchors were originally designed to generate a uniform reaction stress to stabilize and retain the soil mass behind the anchor panels. Figure 5 illustrates in top view the reaction distribution of stress behind the anchor panels and the overlapping zones of stress influence between the anchors by assuming a simplified 1:1 stress dispersion model.

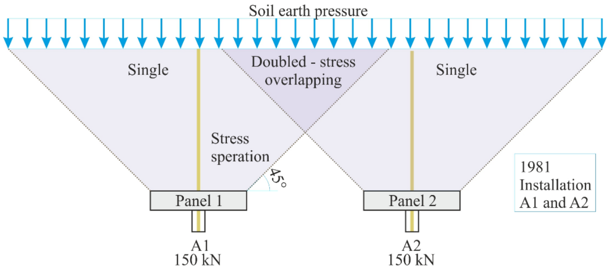

Figure 5. Stress distribution in soil after installing anchors A1 and A2.

In 1997 (16 years after the original installation), it was observed that the anchor forces had increased to A1=280 kN and A2=450 kN, thereby exceeding their design capacities. To mitigate the risk of structural failure and ensure long-term stability for the slope, it was decided to install an additional anchor A3 located between the two old anchors. This measure aimed to redistribute the loads more effectively, transferring the forces from two to three anchors. The load at anchorage (lock-off load) for this anchor was A3=450 kN with designed carrying capacity of 600 kN. In this stage, the resulting stress distribution in the soil behind the anchor panels increased, especially in the middle and right side around anchors A2 and A3, as shown in Figure 6.

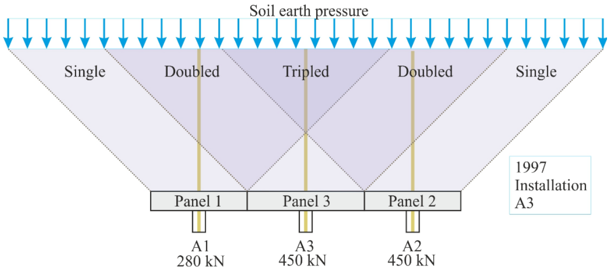

Figure 6. Stress distribution in soil after the installation of anchor A3.

Over the years, the anchor forces continued to increase until 2023, when a sudden drop in the load carried by anchor A1 was recorded, decreasing to 80 kN. Concurrently, the forces in anchors A2 and A3 rose to 528 kN and 581 kN, respectively. One year later, anchor A1 failed completely, without any remaining load contribution, resulting in further increases in the loads carried by anchors A2=575 kN and A3=750 kN. These values significantly exceeded the original design capacities of the respective anchors. This behavior is comparable to the finding of Daxer et al. (2024) that noticed in a 3D numerical simulation and analysis, that the stress redistribution after an anchor failure in an anchor group, caused an increase in load of the adjacent anchors up to 21%.

The failure occurred in anchor A1 after the reaction force exceeded its proof load of 150 kN and reached approximately 300 kN. This overload results in a complete loss of its anchorage function. This unbalanced distribution of loads disrupted the equilibrium of the soil-structure interaction, particularly behind the anchor panels, and led to stress concentrations on one side of the system. The resulting asymmetry in soil pressure between the left and right sides of the slope triggered a local overstressing and ultimately caused cracking and deformation in the slope surface as presented in Figure 7. The resulting stress fields correlate well with the observed crack patterns and provide a logical interpretation of the failure progression.

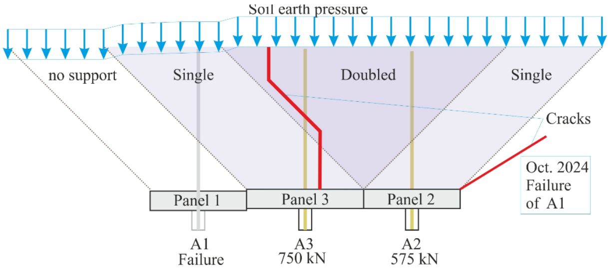

Figure 7. Stress distribution in soil after the failure in anchor A1.

The anchor specifications are listed in Table 1. While the development of anchor loads from the installation of A1 and A2 in 1981 till the failure of A1 in 2024 are present in Figure 8.

Table 1. Specifications and loads of the existing anchorage system.

Anchor	A1	A2	A3
Installation year	1981	1981	1997
System	Bilfinger & Berger		
Type	Permanent		
Permission	Z 20.1-13		Z 20.1-68
Steel	St 1420/1570		St 1570/1770
Number of strands	5	5	5
Strand diameter	12 mm	12 mm	0.6"
Inclination (vertical angle)	16°	16°	16°
Horizontal orientation angle	0	0	0
Total length [m]	8.0	8.0	8.0
Fixed length (bond) [m]	3.0	3.0	3.0
Design anchor load [kN]	400	400	600
Test load (proof load) [kN]	175	400	600
Load at anchorage [kN] (lock-off load)	150	150	450
Anchor load (at A1 failure)	0	575	750

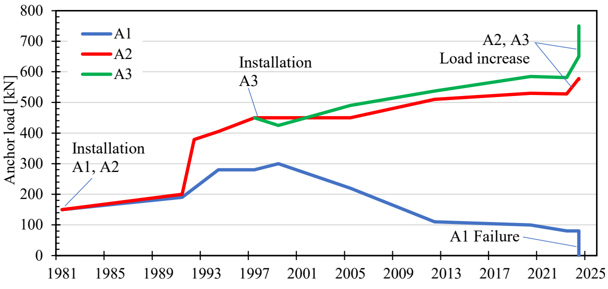

Figure 8. Measured the load of the existing anchors.

3 REMEDIATION

Following the failure of anchor A1 and the rapid progression of slope displacement, emergency stabilization measures became necessary. These interventions were carried out to prevent further slope movement and to maintain structural integrity and safety of the bridge.

In order to prevent a complete failure of the anchoring system, a decision was made to design and implement a compensatory support system that consists of: Additional anchors designed to assist the existing anchorage by redistributing the loads through a steel frame connection; A net of soil nails and a steel wire mesh installed across the embankment surface; And a comprehensive monitoring system to gain a deeper understanding of the long-term performance of the stabilization.

3.1 *Additional anchors*

Four additional anchors were designed to compensate for the complete failure of anchor A1 and to relieve anchor A2, which had reached a load level approaching its ultimate capacity. Furthermore, the installation of these additional anchors aimed to prevent any further load increase on anchor A3, which had already exceeded its design load. According to Sabatini et al. (1999), failures of adjacent anchors should prompt a reassessment of subsurface conditions and/or the design and installation procedures, which may include reducing the design load per anchor by increasing the total number of anchors.

Anchors A4 and A5 were installed 2.0 and 1.5 m to the left of anchor A1, respectively. While anchor A6 was positioned 1.5 m, and anchor A7 was located 2.0 m to the right of anchor A2. Each of the new anchors was designed with an ultimate load capacity of about 600 kN, providing sufficient reserve capacity to support and stabilize the entire anchoring system. These anchors were stressed to a lock-off load of 150 kN over a steel frame to ensure stabilization and redistribution of the stresses related to the global anchorage system. The specifications of the additional anchors are listed in Table 2.

Table 2. Specifications of the additional anchors.

Anchor	A4 and A7	A5 and A6
Installation year	2024	2024
System	Bauer - SPANTEC	
Type	Permanent	Permanent
Permission	Z-34.11-242	Z-34.11-242
Steel	St 1570/1770	St 1570/1770
Number of strands	5	5
Strand diameter	0.6"	0.6"
Inclination (vertical angle)	16°	16°
Horizontal orientation angle	10°	5°
Total length [m]	12.0	10.0
Fixed length (bond) [m]	4.0	4.0
Design anchor load [kN]	600	600
Load at anchorage [kN] (lock-off load)	150	150

3.2 *Steel frame*

The steel frame (belt structure) consisted of two horizontal HEB450 beams and four vertical connection members, onto which the heads of the new anchors were mounted. Load distribution from the new anchors to the existing anchorage system was achieved through this steel frame, which transferred the forces to the existing concrete panels 1 and 2 associated with anchors A1 and A2. Special care was taken to ensure that the concrete panel 3 of anchor A3 remained unloaded to avoid imposing additional stress on its underlying soil, which was already overstressed.

Four hydraulic jacks, each with a capacity of up to 60 tons, were installed between the steel frame and the existing concrete panels 1 and 2. These jacks were used to precisely control and rebalance the load distribution, in addition to compensating for any pressure loss within the entire anchorage system along the lifespan of the structure. The details of the steel frame are depicted in Figure 9.

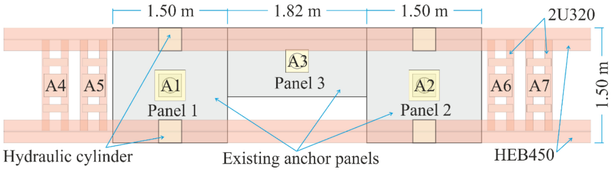

Figure 9. Steel frame – anchor loads transfer platform.

3.3 *Loading steps*

The loading sequence was carefully planned to ensure that the system remained in static equilibrium, thereby preventing any further failure of the slope or disturbance to the existing ground structure.

The initial loading steps were carried out on the outer anchors A4 and A7 at the same time, in increments of 50 kN up to a total load of 150 kN. Subsequently, the same loading procedure was applied to the inner anchors, A5 and A6. Shortly after the lock-off of anchors A5 and A6, a redistribution of loads occurred within the system, resulting in an increase in the load on anchor A7 to 173 kN and a reduction on anchor A5 to 140 kN. To restore balance within the system, hydraulic jacking was performed, primarily on Panel 1 (left side), in order to increase the load carried by anchors A4 and A5. Following this adjustment, the anchor loads stabilized at A4=160 kN, A5=147 kN, A6=146 kN, and A7=169 kN. This final load configuration reflects a symmetrical distribution across the new anchorage system.

After 20 days, a reduction in anchor loads was observed, which was likely due to the strand strain (elongation) in the free length of the anchors. As a result, re-jacking was necessary to restore adequate support loads and to prevent further displacement of the slope. The re-jacking procedure was carried out similarly to the previous adjustment, with a focus on the left side of the system where the load reduction was more pronounced. Following the re-jacking, the loads stabilized at A4=171 kN, A5=162 kN, A6=128 kN, and A7=150 kN. The loading steps are listed in Table 3. The installation of the additional anchors with the jacking system is shown in Figure 10.

Table 3. Loading steps for stressing the new additional anchors.

Anchor	Step	A4 [kN]	A5 [kN]	A6 [kN]	A7 [kN]
Anchors tension	1	50			50
Anchors tension	2	100			100
Anchors tension	3	150			150
Anchors tension	4		50	50	
Anchors tension	5		100	100	
Anchors tension	6		150	150	
Lock-off anchors	7	150	140	149	173
Jacking	8	Panel 1 = 200		Panel 2 = 8	
Load stabilizing and redistribution	9	160	147	146	169
Measurement after 20 days	10	141	123	123	153
Re-jacking	11	Panel 1 = 200		Panel 2 = 8	
Load stabilizing and redistribution	12	171	162	128	150

During the installation of the additional support system and in the subsequent monitoring period, the existing anchors A2 and A3 exhibit slight fluctuations in the recorded load measurements. These variations are likely attributable to temperature changes, which can influence the accuracy of pressure gauges and load cells. Figure 11 shows the measured load for the existing as well as the new additional anchors.

Figure 10. Installation of the additional anchors with jacking system.

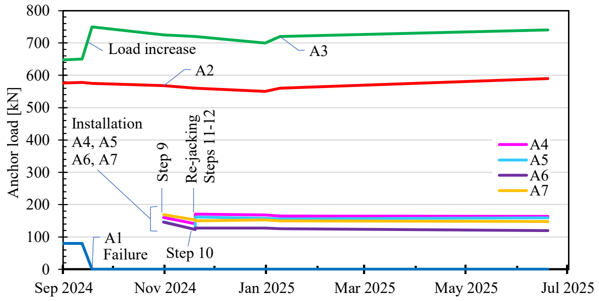

Figure 11. Measured load of the anchors after the installation of the additional anchors.

The four additional anchors were installed and connected to the compensatory steel frame. This frame was supported by the new anchors and braced exclusively against the existing panels 1 and 2 corresponding with anchors A1 and A2. Using hydraulic jacks, particular emphasis was placed on reactivating the soil reaction behind the left-side panel 1, thereby rebalancing the load distribution and restoring stress equilibrium behind the anchorage system. As illustrate in Figure 12, a redistribution of the stress behind the anchor panels is ensured, which has a considerable contribution to the reduction of the stress asymmetry behind the panels.

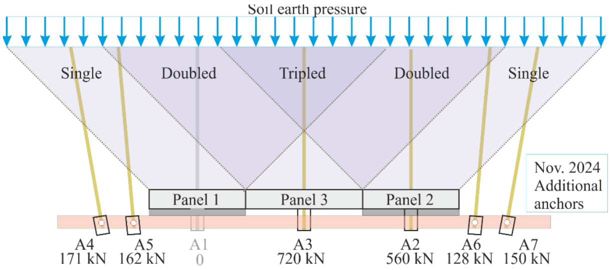

Figure 12. Stress distribution in soil after installing the additional anchors A4, A5, A6 and A7.

The supporting additional anchorage system has the capacity to support more increments in load and redistribute the associated stress on the both sides for the long-term lifespan of the structure. Furthermore, any stress unbalanced in the system can be controlled and adjusted via the hydraulic jacks.

3.4 *Soil nailing and steel mesh*

To enhance slope stability and prevent further progression of slip or shear failure, a system of soil nailing was implemented. Soil nailing is a ground reinforcement technique that involves the insertion of steel rods (nails) into the soil mass to increase its overall stability. This method mobilizes frictional resistance along the length of the nails, thereby enhancing the shear strength of the slope. The relevant technical local guidance is given in DIN EN 14490 (2010) and EBGEO (2010), which outline best practices for the use of geotechnical methods in road construction and slope remediation.

The soil nails were combined with a high-tensile steel mesh installed across the surface of the embankment. This mesh facilitates the distribution of loads between the nails over the surface area of the embankment, thereby providing surface stability and preventing shallow slips. In addition, it serves as a protective barrier, preventing cobblestones from detaching and falling onto the highway below, as shown in Figure 13.

Figure 13. Soil nail with steel mesh.

In parallel, the visible damages and deformations were repaired. Specifically, the cobblestone surface and the steps of the stairway were restored to reinstate both serviceability and aesthetic appearance. The remediation includes the additional anchors and soil nails depicted in Figure 14 and Figure 15.

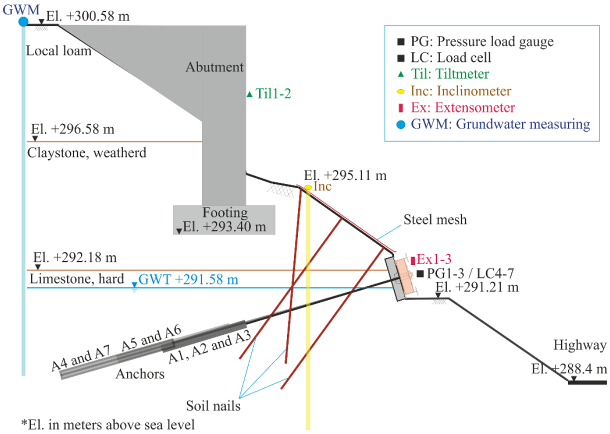

Figure 14. Side view – remediation measures and monitoring system.

3.5 *Monitoring*

To ensure long-term performance of the stabilization measures and to detect any signs of renewed displacement or increment of stress, a comprehensive monitoring system was implemented. The system was designed to observe and record key parameters such as anchor loads, slope displacements, and potential deformations in the abutment. Integrated monitoring systems are also increasingly employed to ensure real-time observation of structural and geotechnical behavior.

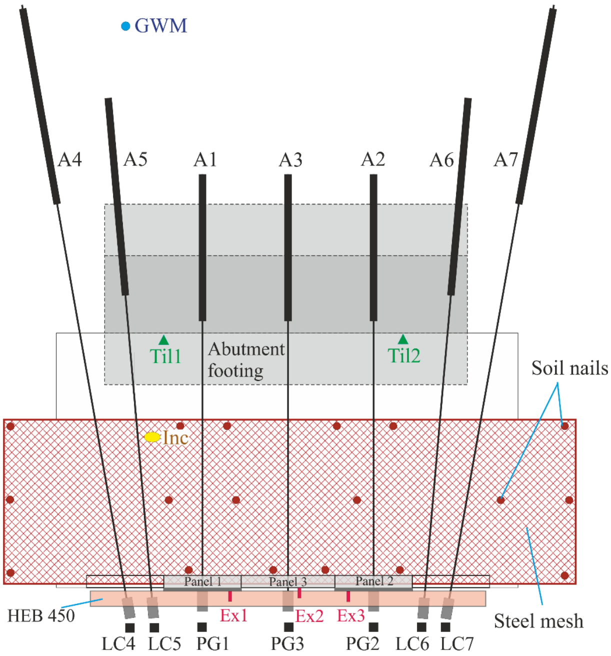

Figure 15. Top view – remediation measures and monitoring system.

The instrumentation includes: Load cells installed at the heads of the newly added anchors to continuously measure anchor forces; An inclinometer, was used in a vertically cased borehole, recorded lateral ground movements at various depths of the embankment; Extensometers were used to monitor the displacement of the existing anchor panels along the direction of the anchor strands; Tiltmeters used on the abutment to detect rotational movements around two axes; And a groundwater monitoring well implemented to observe real-time changes in the groundwater table, which could affect slope stability. Figure 14 and Figure 15 display the location of the sensors over the embankment and the anchorage system. Figure 16 displays a view after the implementation of remediation measures.

Figure 16. General view for the embankment after the remediation.

4 RESULTS

The newly installed anchors (A4–A7) effectively carried the reaction loads, fully compensating for the failed anchor A1. Since then, no significant change in anchor forces has been observed; therefore, the load distribution has remained stable and within the design limits, as it is clearly noticed in Figure 11.

Inclinometer data revealed that within the first month following the remediation, the slope experienced a lateral displacement parallel to the highway direction of approximately 4.5 mm. Thereafter, the rate of movement decreased significantly. Within the subsequent measurements, an additional displacement of about 1 mm takes place, as shown in Figure 17.

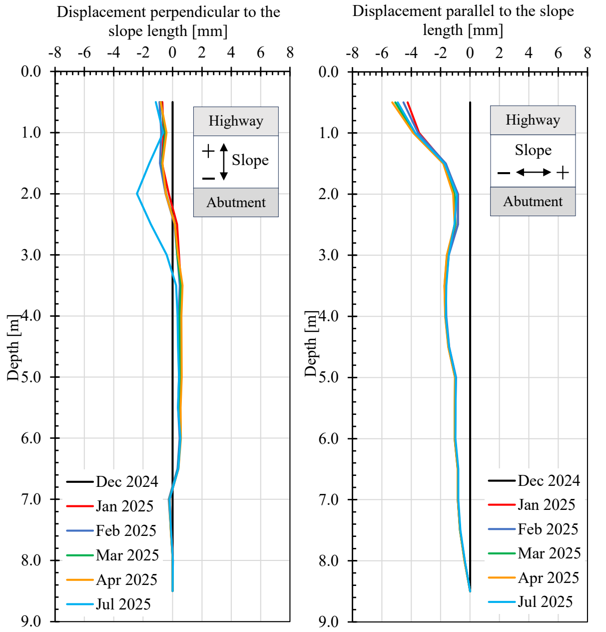

Figure 17. Slope deformation in both directions – inclinometer results.

As in Figure 18 and Figure 19 present, the tilt sensor measurements indicated no significant rotational movement of the bridge abutment around both axes. Similarly, displacement measurements obtained from the extensometers showed no detectable movement of the anchor panels in the direction perpendicular to the embankment.

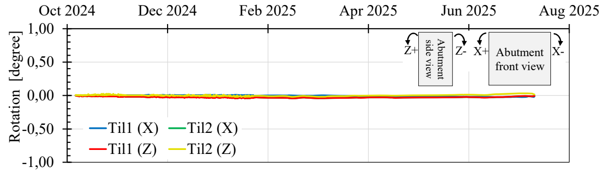

Figure 18. Rotation of the bridge abutment – tiltmeter results.

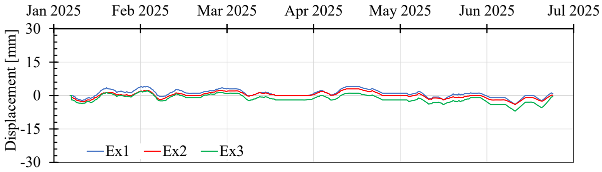

Figure 19. Displacement of the anchor panels – extensometer results.

The measured groundwater table reached a maximum level approximately at the elevation of the anchor heads, as shown in Figure 20. This indicates that an increase in anchor loads or structural damage due to rising water pressure is unlikely. The observations also confirm that the existing drainage pipe, located above the anchor heads, is still functioning effectively and maintaining proper drainage conditions.

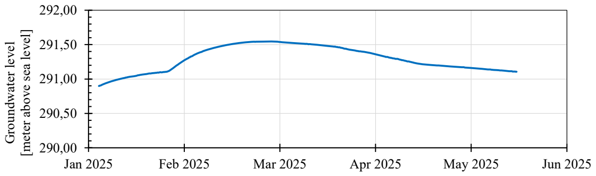

Figure 20. Groundwater level – groundwater monitoring well.

The consistent results from the monitoring system confirm that the slope remains in a stable condition. Furthermore, no additional visible cracks or structural damages have been observed on the embankment surface or the stairway.

5 CONCLUSIONS

The presented case study addressed a slope stabilization challenge where progressive ground movement, and anchor failure and overloading posed a significant threat to the safety and functionality of a bridge structure.

The implemented remediation measures, including additional anchors, a steel frame, soil nailing with steel mesh, and a comprehensive monitoring program, have provided effective slope stabilization and restored the structural safety of the bridge and its surrounding infrastructure. Monitoring data confirmed that anchor loads were successfully redistributed, progressive deformations ceased, and no further structural damage or visible surface cracks developed.

From an economic standpoint, the selected approach demonstrated high cost-efficiency compared to more invasive alternatives, such as complete embankment reconstruction or replacement of the bridge substructure. This solution avoided extensive demolition and reconstruction efforts and was implemented without closures or operational restrictions on the adjacent highway, thereby minimizing both direct construction costs and indirect logistical impacts. The adopted remediation strategy is highly practical, making it suitable for emergency cases, whereas a rapid construction period of one to two months is necessary. It is also transferable to similar cases involving aging infrastructure affected by anchor failure or slope instability. By relying on conventional materials and methods, the solution requires no specialized equipment or technologies.

Overall, the applied measures proved to be efficient, economical, and practical, offering a reliable solution for similar geotechnical challenges.

6 ACKNOWLEDGEMENTS

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