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1915 Çanakkale Bridge European anchor block excavation project: a case study

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ABSTRACT

This paper presents a deep excavation of the reinforced concrete anchor block structure with a volume of 68,000 m³, located on the European side of the 1915 Çanakkale Bridge. Slope stability, excavation, and dewatering works within the scope of the European Anchor Block Project with a maximum excavation depth of 46,0 m were completed within the tight schedule in a very heterogeneous soil profile consisting of Miocene-based sandstone-mudstone layers and Holocene deposits containing gravelly sand-clay units, especially on the seaside. Within the scope of the project, 170,000 m³ of excavation work, 10,000 m of rock bolts, 12,000 m² of shotcrete applied with ±1cm construction tolerance, besides 431 no's of secant piles and 28 no's of dewatering wells were executed. In order to verify the in-situ design capacity of the rock bolts, a total of 26 no's of preliminary pull-out tests were performed, on each side of the excavation pit and each excavation stage prior to the commencement of the production rock bolts. In this study, the design of the slope stabilization project, details of the construction works, and the results of the rock bolt tests are presented. Detailed information about the quality control tests followed within the scope of the project and the performance of the slope protection system has also been presented.

KEYWORDS

Slope stability, Deep Excavation, Rock bolt, Pull-out test, Quality control tests.

1. INTRODUCTION

The 1915 Çanakkale Bridge, with a main span of 2,023 meters, commemorating the centenary of the founding of the Republic of Turkey, has become the largest mid-span suspension bridge in the world, surpassing the Akashi Kaikyo Bridge in Japan. The 1915 Çanakkale Bridge has a total length of 4,608 meters, with a central span of 2,023 meters, side spans of 770 meters, and approach viaducts of 365 and 680 meters. The approach viaduct on the Asian side (Lapseki) of the 1915 Çanakkale Bridge is 680 meters long, and the approach viaduct on the European side (Gallipoli) is 365 meters long. The anchor block is to comprise two front pads and a rear concrete massif. The rear concrete massif is to be 16.0m deep (below natural ground level), 50.2m wide, and 74.4m long, and the front anchor pads are to be 18.0m by 17.5m in plan and 5.0m deep (below ground level). Each front pad is to be connected to the rear massif with a connecting beam. The project map photo of the study area is given in Figure 1. The excavation geometry for the anchor block is mainly defined by the distance between the pier PE1 and the abutment, which is defined as 51 meters horizontally. The maximum excavation depth is approximately 46 meters. Within the scope of the study, detailed information about anchor block excavation, the design of the shoring system, monitoring of shoring performance, quality control test results, and production details are presented.

Figure 1. Map photo of the study area.

2. SITE CHARACTERIZATION

Detailed soil investigation was carried out at the project site by Fugro. 4 boreholes with depths ranging from 30-70 m and 6 CPT tests with depths ranging from 10.6 m to 35.70 m were performed. Soil classification was made using the Robertson 1990 classification chart by taking into account results obtained from CPT experiments. The idealized soil profile is given in Figure 2.

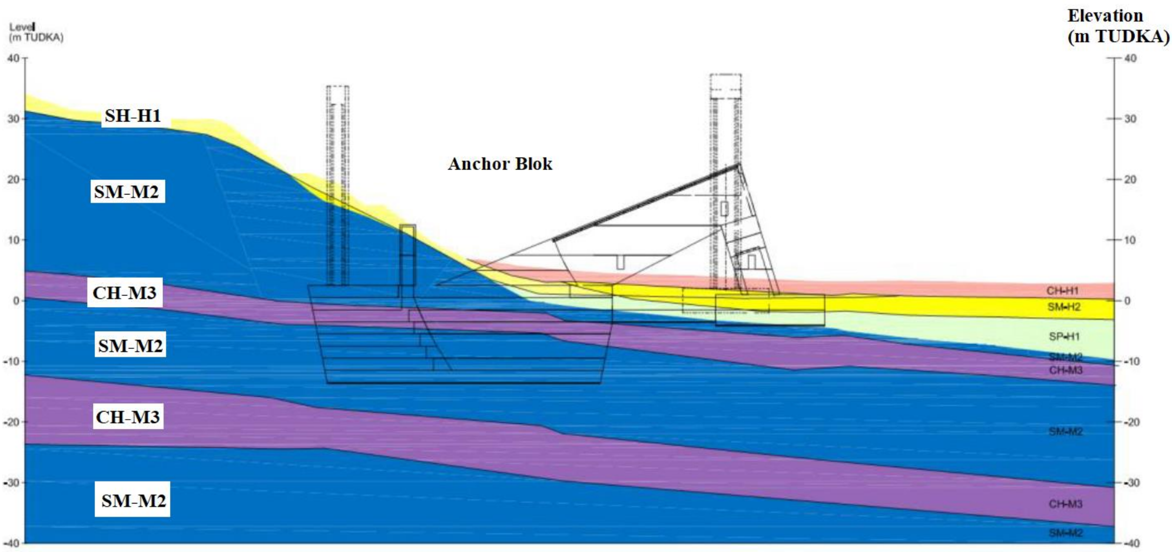

Figure 2. Idealized soil profile.

Generally, thickness of the Holocene deposits; described as mixed layers of lean, sandy to fat clay (CL-H1 to CH-H1) to fine, silty sand (SM-H1/-H2) to well-graded, gravelly sand (SP-H1) is significantly

larger on the European side, than on the Asian side, while the Miocene basement mainly comprises Sandstone, SM-M2, embedded with layers of extremely weak to weak mudstone, CH-M3.
Geotechnical design parameters of soil layers are given in Table 1.

Table 1. Geotechnical design parameters of soil layers.

Soil layer	γ [KN/m³]	w [%]	ϕ' [°]	c' [kPa]	c_u [kPa]	E [MPa]	ν_u	C_c [-]	C_r [-]	k [m/s]
CL-H1	19	28.5	29	0	40	-	0.5	0.15	0.02	$5 \cdot 10^{-9}$
CH-H1	19	24	29	0	40	-	0.5	0.15	0.02	$5 \cdot 10^{-9}$
SM-H1	18	30	30 1)	0	-	6	0.5	-	-	$5 \cdot 10^{-5}$
SM-H2	19	25	33 1)	0	-	24	0.5	-	-	$5 \cdot 10^{-5}$
SP-P1	19	18.5	35	0	-	37	0.5	-	-	>0.001
SM-M2	21	17	37	300	-	-	0.5	-	-	-
CH-M3	22	10	40	40	600	-	0.5	-	0.008	-

3. ANCHOR BLOCK EXCAVATION

The general layout plan of the project site is given in Figure 3. The excavation geometry for the anchor block is mainly defined by the distance between the pier PE1 and the abutment, which is horizontally, 51 m as shown in Figure 4. The maximum excavation depth is 46 m approximately.

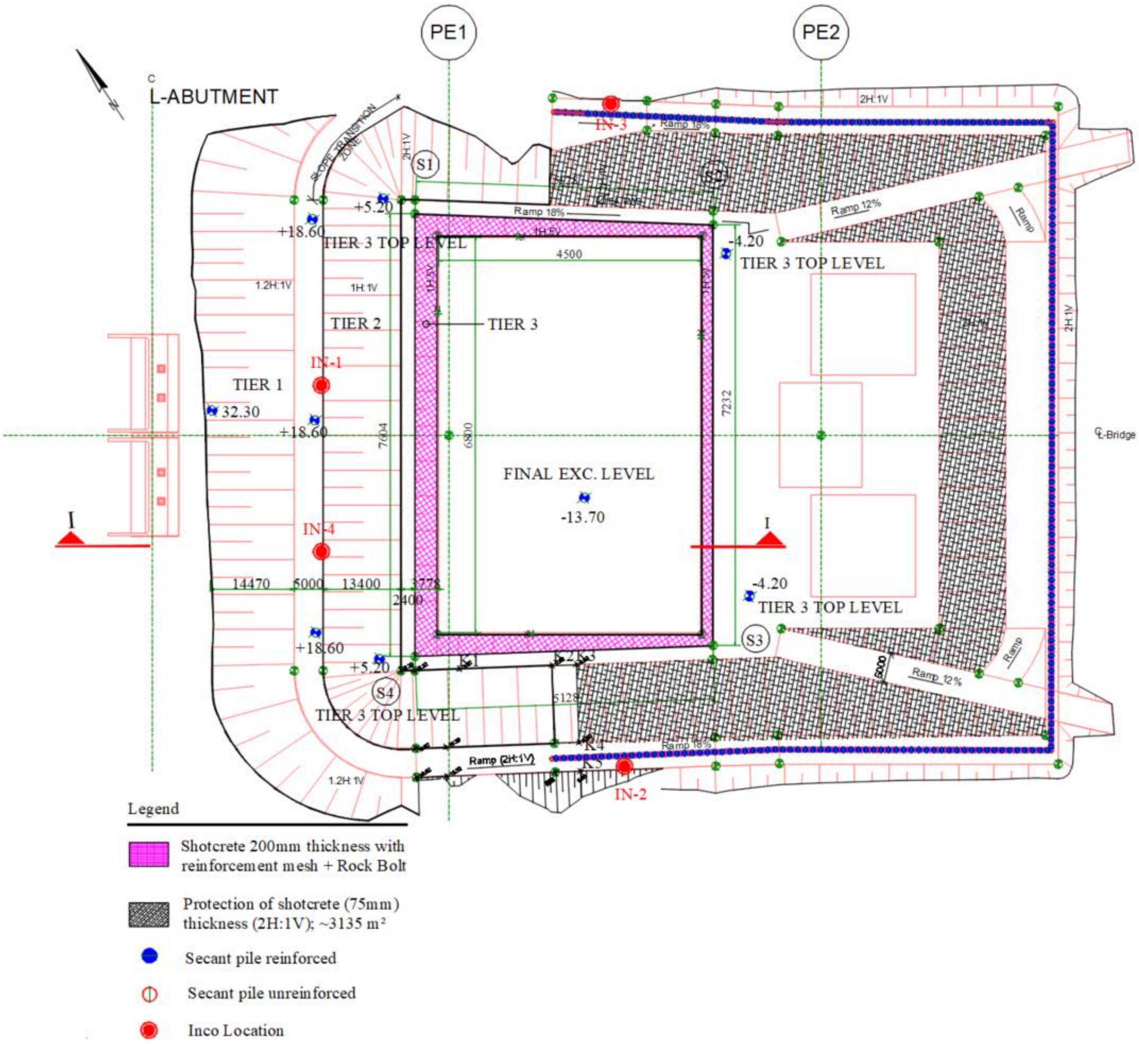

Figure 3. General layout plan.

The slope of the upper section is 1.2H:1V with a height of 13.7 m; the slope of middle section is 1H:1V with a height of 13.4 m and and the slope of bottom section is 1H/5V with a height of 18.9 m. A U-shaped secant pile wall on the sea side of the excavation is designed in order to cut-off water inflow from the Holocene deposits and Sandstone layers into the excavation area. The toe level of the secant pile wall is -8.00 m, in such a way that the piled wall reached the mudstone level.

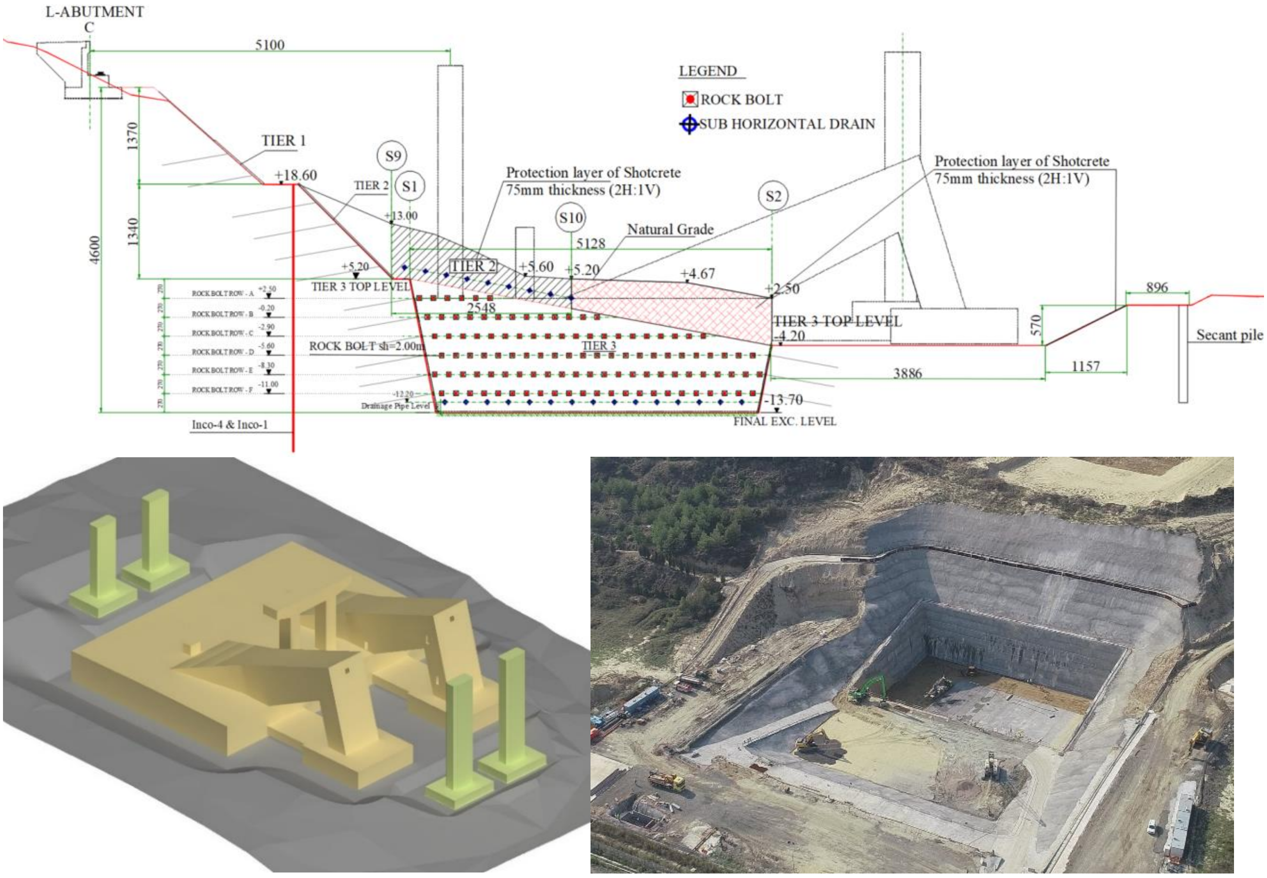

Figure 4. Excavation section and 3D view of anchor block.

3.1. Safety factors for global stability

Conservatively, the global stability calculations have been performed considering the unit SM-M2 with a reduced effective cohesion (see Figure 5) due to possible weakened zones in the soil mass and a 6 m thick layer close to the excavation face with the residual strength properties of the sandstone to consider the weakening of the sandstone due to the unconfinement and weathering effects during the excavation. The analysis of the slope stability for the excavation works at the anchor block has been carried out using PLAXIS 2D. For verification of the global stability, the specified minimum safety factors are given in Table 2, according to the KGM Technical Specification (both for temporary and permanent structures).

Table 2. Safety factors for global stability analysis

Soil Parameter/Condition	With undrained strength parameters	With Effective strength parameters
Long term analysis (Static)	-----	1.5
Short term analysis (Static)	1.5	-----

The values defined in KGM specifications are total factor of safety for analyses performed with characteristic soil parameters ($\gamma c' = \gamma \varphi' = \gamma cu = 1.0$). Effective strength properties have been considered only for the evaluation of the stability of the excavation due to the nature of the prevailing soil conditions. The finite element analysis model, maximum displacement at the final excavation stage, and critical shear envelopes are summarized in Figure 5.

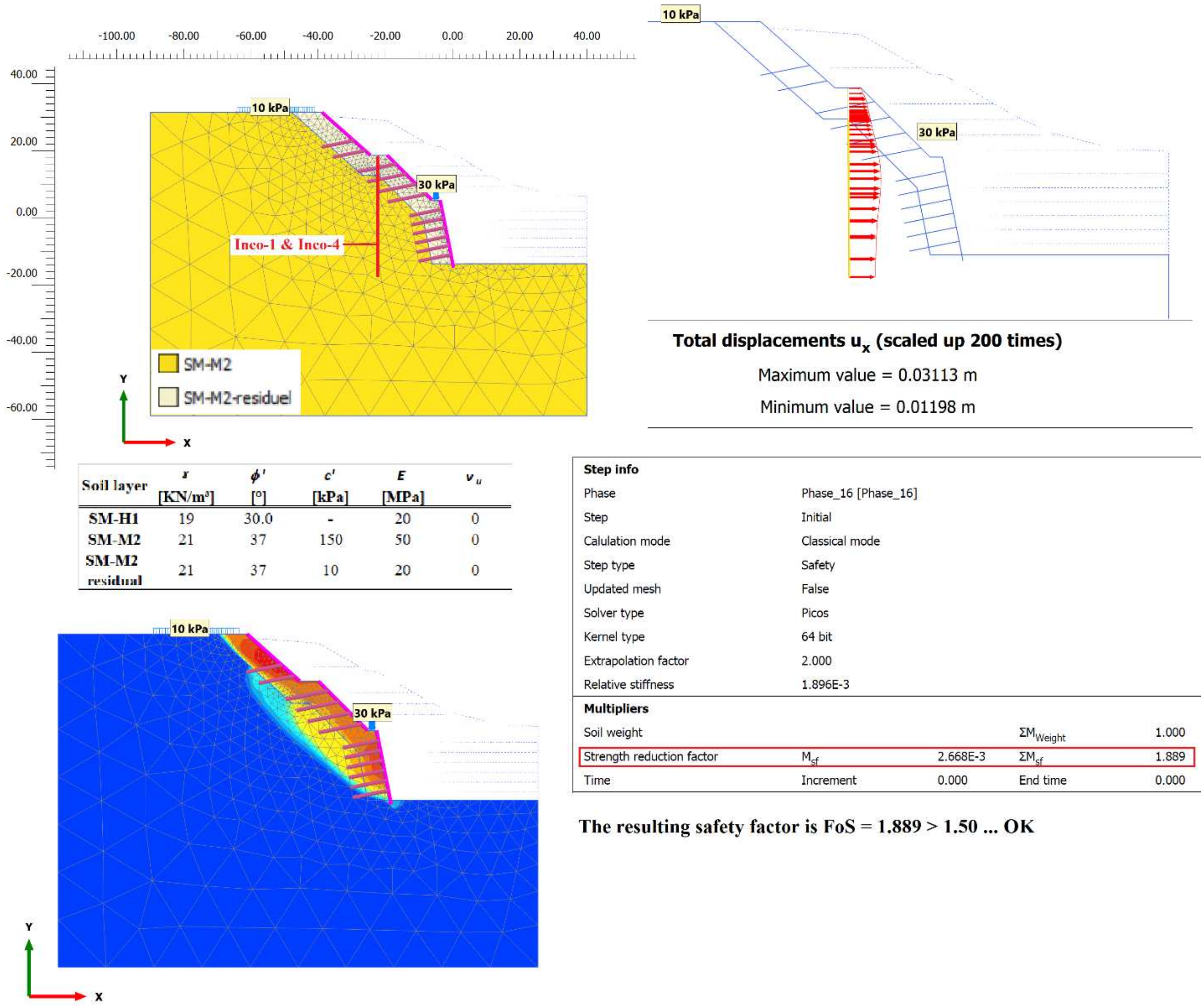

Figure 5. Plaxis 2D analysis summary.

As a result of the finite element analysis, the safety factor against total collapse was estimated to be 1.89.

In the final excavation stage, the variation of the displacement amount obtained with the Plaxis finite element software with depth and the displacement values obtained from the inclinometer boreholes No:1 and No:4 are given in Figure 6. As can be seen from the graph below, the calculated (estimated) displacement values and the actual displacement values were very close to each other.

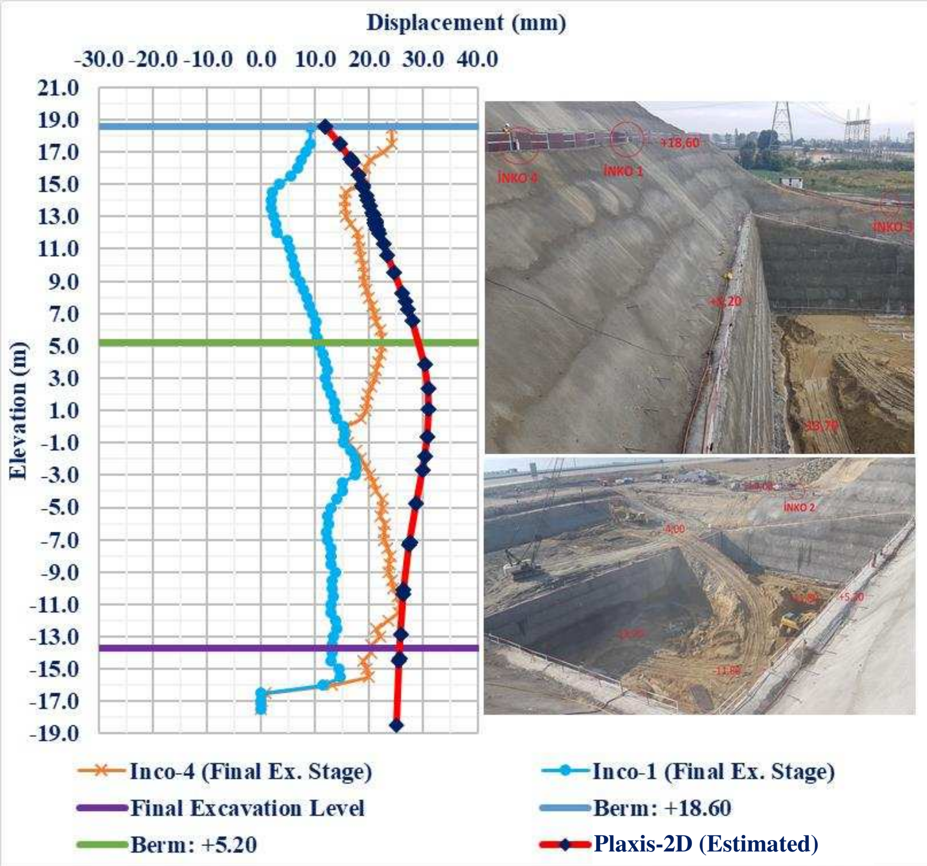

Figure 6. Estimated and actual displacement

4. ROCK BOLT PULL-OUT TESTS

The slope stability of the excavation is designed by shotcrete and grouted rock bolts drilled into the rock (sandstone), as shown in Figure 4. In order to calculate the maximum pull-out resistance per grouted bar, lateral skin resistance has been estimated as follows:

$$T_{skin,max} = \pi x \emptyset_{drill} x \tau \tag{1}$$

where, $\emptyset_{drill}$, is the diameter of the drilling, which is defined as $\emptyset_{drill} = 76.2mm$. τ, represents the characteristic soil/grout bond strength. The estimated value for the bond strength τ in the Sandstone can be obtained from Table 3, assuming conservatively an upper value of the bond strength for weathered sandstone proposed in the table. This bond strength is considered to be a conservative/lower bound value for the inner "intact" sandstone. For the verification of the pull-out force, a bond strength of τ=300kPa has been assumed.

Table 3. Bond strength estimation for rock bolts in rock (Lazarte 1 et al., 2003).

Material	Construction Method	Soil/Rock Type	Ultimate Bond Strength, qu (kPa)
Rock	Rotary Drilled	Marl/limestone	300 - 400
		Soft dolomite	400 - 600
		Fissured dolomite	600 - 1000
		Weathered sandstone	**200 - 300**
		Weathered shale	100 - 150
		Basalt	500 - 600
		Slate/Hard shale	300 - 400

The ultimate characteristic pull-out force per meter of rock bolt is, then:

$$T_{skin,max} = \pi x \emptyset_{drill} x\tau = \pi \text{x} 0.0762 \text{x} 300 = 71.82 \text{ kN/m}$$

Since the EN1997-1 does not consider the design of the soil nails or rock bolts a reasonable partial factor on pull out resistance of γ=1.50 has been adopted following the BS 8006-2.

$$T_{skin,max,d} = \frac{T_{skin,max,d}}{\gamma} = \frac{71.82}{1.5} = 47.88 \; kN/m \quad (2)$$

Nevertheless, the maximum bond strength was verified during the pull-out tests to be carried out on-site before the excavations. The maximum force in the rock bolt will be limited by the maximum tensile capacity of the steel bar, as follows:

$$F_{y,d} = \frac{F_{y,k\,y}}{\gamma} = \frac{500x10^3}{1.15} x \frac{\pi x 0.032^2}{4} \quad (3)$$

4.1. Sacrificial Tests

26 pull-out tests were performed within the scope of the Sacrificial testing program. The location of these tests in the project site is given in Figure-7.

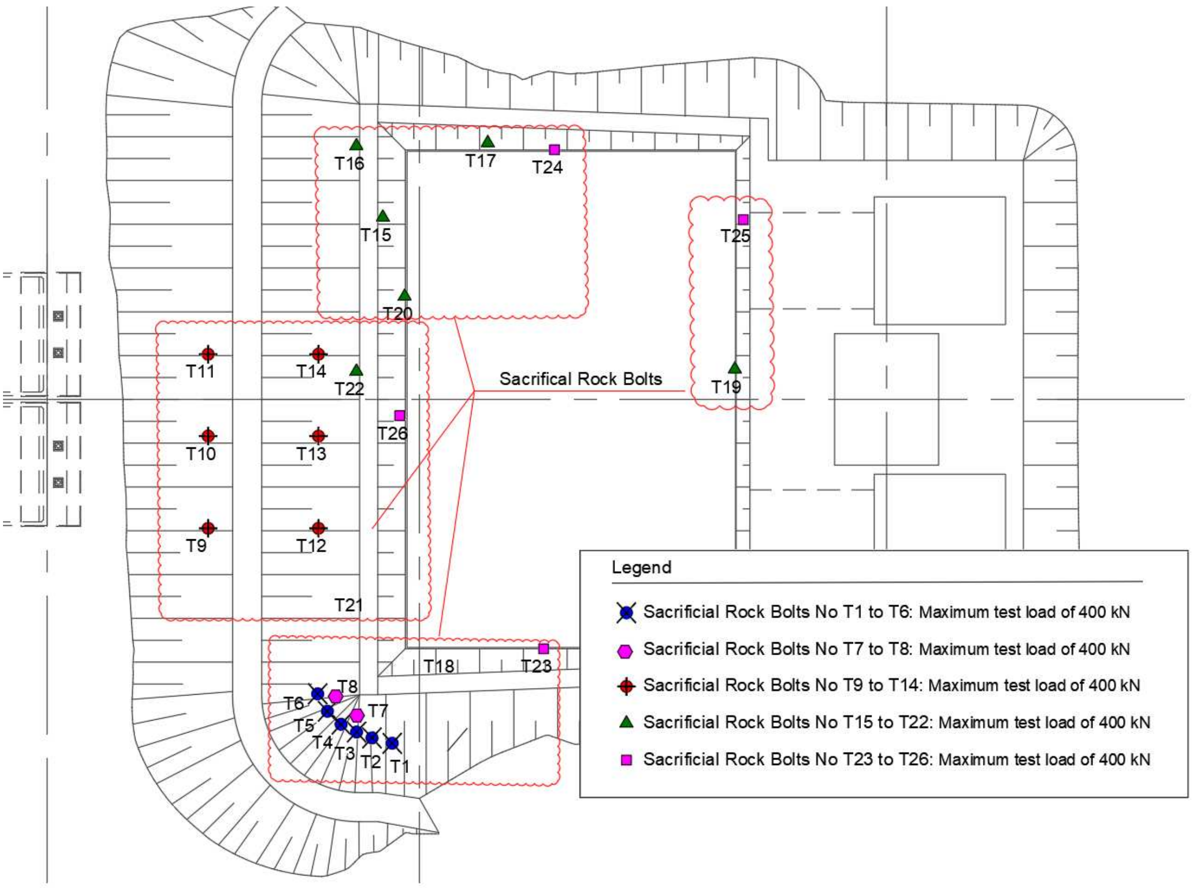

Figure 7. Rock bolt pull-out tests layout plan.

For the all-sacrificial tests, maximum test load of 400 kN was achieved successfully without pull out failure of bond. By considering third cycle of sacrificial rock bolt test for T8, no bond failure was observed until 450-460 kN load of which was near yielding limit of reinforcement. The mobilized bond stress (without failure) of sacrificial rock bolts is given in Table 4.

Table 4. Mobilized Bond Stress (without failure) of Sacrificial Rock Bolts.

Rock Bolt No	Dia. (mm)	Datum Load (kN)	Max. Load (kN)	Elongation Difference (mm)	Permanent Displacement (mm)	Theoretical Displacement (mm)	Apparent Free Length (m)	Rock Bolt Length (m)	Actual Bond Length (m)	Mobilized Bond Stress (kPa)
T1	76.2	40	400	19.2	3.8	15.4	6.5	12.0	5.5	303.8
T2	76.2	40	400	20.2	2.0	18.2	7.3	12.0	4.7	355.5
T3	76.2	40	400	19.1	0.7	18.4	7.4	12.0	4.6	363.2
T4	76.2	40	400	21.7	1.0	20.6	8.3	12.0	3.7	451.6
T5	76.2	40	400	19.4	0.7	18.7	7.5	12.0	4.5	371.3
T6	76.2	40	400	21.1	1.2	19.9	8.0	12.0	4.0	417.7
T7	76.2	40	400	19.8	-1.3	21.1	8.3	12.0	3.1	539.0
T8	76.2	40	460	-	-	25.0	-	12.0	3.3	591.2
T9	76.2	40	400	-	-	20.3	-	12.0	3.9	434.0
T10	76.2	40	400	-	-	20.3	-	12.0	3.9	434.0
T11	76.2	40	400	-	-	20.3	-	12.0	3.9	434.0
T12	76.2	40	400	-	-	20.3	-	12.0	3.9	434.0
T13	76.2	40	400	-	-	20.3	-	12.0	3.9	434.0
T14	76.2	40	400	-	-	20.3	-	12.0	3.9	434.0
T15	76.2	40	400	14.5	3.3	7.5	5.0	10.0	7.0	238.7
T16	76.2	40	400	23.4	-0.6	26.6	10.7	12.0	1.3	1285.3
T17	76.2	40	400	19.8	1.2	15.7	8.3	10.0	3.7	451.6
T18	76.2	40	400	-	-	16.8	-	10.0	3.3	514.1
T19	76.2	40	400	-	-	16.8	-	10.0	3.3	514.1
T20	76.2	40	400	-	-	16.8	-	10.0	3.3	514.1
T21	76.2	40	400	-	-	20.3	-	12.0	3.9	434.0
T22	76.2	40	400	-	-	20.3	-	12.0	3.9	434.0
T23	76.2	40	400	20.6	8.2	8.7	5.5	10.0	6.5	257.1
T24	76.2	40	400	19.6	3.1	13.4	7.4	10.0	4.6	363.2
T25	76.2	40	400	22.5	4.1	15.4	8.2	10.0	3.8	439.7
T26	76.2	40	400	18.4	1.1	14.2	7.7	10.0	4.3	388.6
Average Mobilized Bond Stress (kPa)										**455.1**

During testing of rock bolt; T7, it was observed that elongations based according to measured displacement in both cycles were near to theoretical values. However, displacements and related elongations at both cycles were far away from the theoretical values for rock bolt no T8. This issue can be explained with the fact that datum load of 160 kN, applied for steel plate to set on shotcrete face, was higher than the datum load specified in the test procedure. For this reason, the calculations in Table-4 for determination of actual bond length do not consider rock bolt no T8. Related with this issue, creep performance under maximum test load was evaluated for only rock bolt no T7 and it is concluded that creep rate below the limit defined in relative standard (EN 14490:2010). Having reviewed mobilized bond stress without bond failure and observation of no bond failure at nearly yielding point of reinforcements for T8, T18 to T22, it is clearly seen that installation of production rock bolts with diameter of 76.2 mm will be adequate for assumed maximum bond stress (300 kPa) and maximum design loads for static condition (270 kN per rock bolt). The variation of mobilized bond stress for weathered sandstone according to actual rock bolt lengths is given in Figure 8.

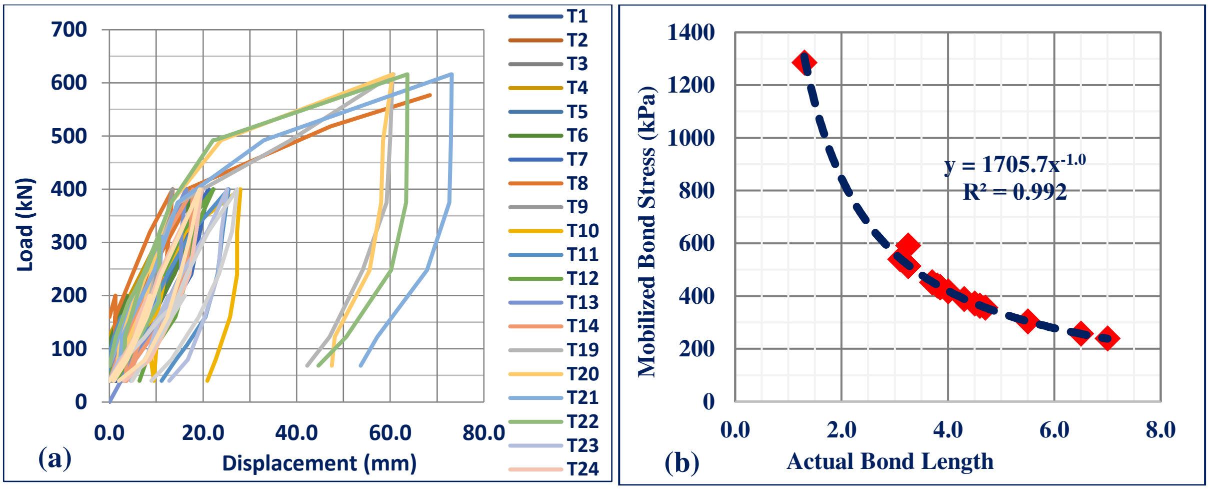

Figure 8. Load-displacement curve (a) and variation of mobilized bond stress with actual bond length (b) for weathered sandstone.

$$\sigma_{mob} = \mathbf{1705.7}/L_{bond} \quad \mathbf{(4)}$$

5. CONCLUSIONS

This article delves into the extensive excavation of the reinforced concrete anchor block structure, boasting a volume of 68,000 m³, situated on the European side of the 1915 Çanakkale Bridge. In the scope of the project, 170,000 m³ of excavation work, 10,000 m of rock bolts, 12,000 m² of shotcrete application, 431 no's of secant piles, and 28 no's of dewatering wells were executed successfully. The amount of displacement obtained in the finite element analysis performed with residual soil parameters was very closely related to the inclinometer readings (Inco-4 and Inco-1) made in the final excavation stage. To verify the in-situ design capacity of the rock bolts, a total of 26 no's of preliminary pull-out tests side of the excavation pit, and results are summarized. Design of temporary excavation works was performed by assuming rock bolts having pull-out resistance of 300 kPa with borehole diameter of 76.2mm. All the sacrificial rock bolts were produced with a borehole diameter of 76.2 mm. All rock bolt tests successfully met the test load without creep.

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