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# Determination of soil nail capacity in greywacke formation

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ABSTRACT: A research study is presented in which the performance, pullout capacity and design parameters of permanent soil nails in the greywacke rocks of the Istanbul are investigated. In order to determine the failure mechanism of the soil nails, pullout tests have been performed on 6 of the soil nails. For this specific project, the critical failure mechanism for the soil nails have been determined as yielding of the rebar. The critical, long term axial capacity of the rebar have been determined using equations and methods that estimate the corrosion rate. The corrosion calculations estimate the reduction of 32 mm rebar diameter from 32 mm to 28.3 mm and the corresponding allowable axial tensile capacity as 140 kN.

## 1 INTRODUCTION

Shoring structures that have a service life in the range of 30 to 100 years are classified as long term or permanent structures. Long term design parameters and higher factors of safeties are used for such long term structures. For the design of permanent soil nailed walls, the internal and external stability of the system is affected by the long term pullout capacity of the soil nails. The long term nail capacity is affected by the effective diameter of the rebar. The diameter of the rebar in embedded structures are reduced in time due to corrosion. There are two approaches that address the corrosion effects of the rebar in permanent soil nail applications. The first approach is to prevent the corrosion effects by applying coating or encapsulation to the rebar. The second approach is to accept the effects of corrosion and to apply a sacrificial thickness to the required rebar diameter. Romanoff (1957), Elias (1990) and Anderson (1996) have developed several different approaches to estimate the corrosion rates. In this paper, the effective rebar area and the long term pullout capacity of soil nails in greywacke formations of Istanbul are investigated, using the second approach mentioned above.

## 2 PROJECT DATA

The investigated project is in the Umraniye district of Istanbul. A soil nailed wall with variable height from 4 m to 11 m and a length of 280 m has been constructed. The wall retains a sloped field of 15 % grade. The service life of the permanent soil nailed wall has been chosen as 70 years.

### 2.1 Geological Conditions

The investigated project site has the predominant greywacke formation of Istanbul which is a claystone-sandstone formation of the Trakya Formation. Greywacke is a common sedimentary rock of Istanbul, that have been investigated in many deep excavation projects. Among the 13 soil borings, brown to light brown greywacke formation has been encountered in almost all of the borings, from top to the bottom. The greywacke formation in this site belongs to the Lower Carbonification Era, with light to normal weatheration degree and normal fractured structure. The average RQD (Rock Quality Designation) value is around RQD = 13. Point load tests performed on the samples obtained from different depths indicate values in the range of $I_s$ = 1.8 - 11.2 MPa.

2.2 Construction of the Soil Nailed Wall

The soil nailed wall is constructed in stages, from top to bottom. In this shoring method, a reinforced earth/rock structure (similar to a fill type “Reinforced Earth Wall”) is created in natural earth mass, using the soil nails. The wire mesh and the shotcrete facing act both as “face stabilizing temporary wall” and “permanent wall” element. The reinforcement elements are usually standard rebar elements and the facing elements are wire mesh and shotcrete (Figure 1). In permanent shoring applications, the protection of the anchor elements (in our case : rebar type soil nails) against corrosion is a very important issue. In this project, in addition to the “sacrificial thickness” applied to the rebar, the rebars are coated with antirust paint and effort is spent to make sure that the rebars are in the center of the drilled hole, protected by the grout filling the annular space.



Figure 1. Soil Nail Elements (The rebar and the wire mesh – shotcrete facing)

2.3 Design of the Soil Nailed Wall

The common failure mechanisms of soil nailed walls are briefly shown in Figure-2. The soil nailed wall either fails through an external failure surface (external or global stability, Figure 2 a) or through internal failure surface (internal stability, Figure 2 b, c). The internal failure mechanism, can either occur from an internal sliding of the nail-grout system from the soil or from an internal sliding of the rebar from the grout mass (Figure 2 b). Another internal failure mechanism is the rupture of the nail-grout system under tension and bending (Figure 2 c). All three mechanisms have to be investigated in the design.

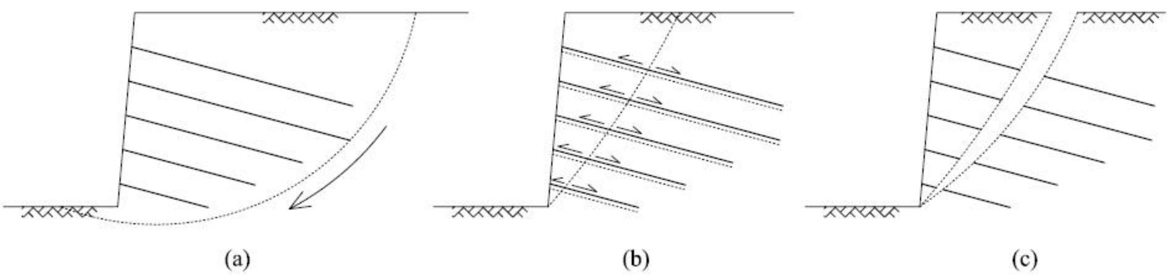

Figure 2. Possible failure mechanisms in a soil nailed wall

Assuming that the mechanical characteristics of the grout mass and the rebar are relatively controllable and can be kept constant, the frictional resistance between the grout and the soil/rock is the critical parameter of the internal sliding failure. The frictional resistance between the grout and the soil/rock depends mostly on the strength parameters of the soil/rock formation, plasticity and gradation of the soil. The drilling method and the grout injection pressure also affect the frictional resistance between the grout and the soil/rock. The frictional resistance between the grout and the rebar is increased using corrugated rebars. The corrugated surface structure of the rebar creates a mechanical interlock between the rebar and the grout. One of the major objectives in this project was to determine the critical internal failure mechanism for soil nails in the greywacke formation by carrying out pullout tests.

## 3 DETERMINATION OF THE ULTIMATE TENSILE CAPACITY FROM THE PULLOUT TEST

The pullout test have been carried out in 6 of the soil nails with 32 mm diameter and variable lengths (L = 4, 7 and 10 m). The deformations corresponding to different load levels have been measured in the test. The tests have been carried out until failure.

### 3.1 Test Equipment and Procedure :

The test have been carried out on nails that have a minimum grout set of 14 days. Tensile load have been applied to the rebar element through the use of a hydraulic piston. The hydraulic piston has been placed on a tripod steel carriage unit that rests on the ground. The hydraulic piston applies compression reaction to the plate that rest on the shotcrete face and the same compression reaction load is applied as tension load to the nail head. The deformation of the nail head and the nail plate are measured via two separate dial gauge. The top 1 m embedded portion of the soil nail has not been grouted in order to maintain pure tension load on the nail element. The equipment used in the test are as following (Figure 3) :

- Hydraulic piston (hollow system) with 80 ton loading capacity;
- Tripod steel carriage unit;
- 2 Loading plates;
- 2 Dial Gauge with 0.01 mm precision.

Figure 3. Photo of the Test Apparatus during the test.

### 3.2 Test Method

The standard "controlled displacement test" have been modified as "controlled rapid load test" "Clouterre (1991)". The load has been applied until failure with 25 % increments of the project load.

The load durations at each load level has been chosen as 5 minutes. The displacement of the free nail head and the shotcrete face have been measured separately with the 2 dial gauge.

3.3 Test Results

The load- displacement curve of the 6 pull out tests have been summarized in Figure 4.

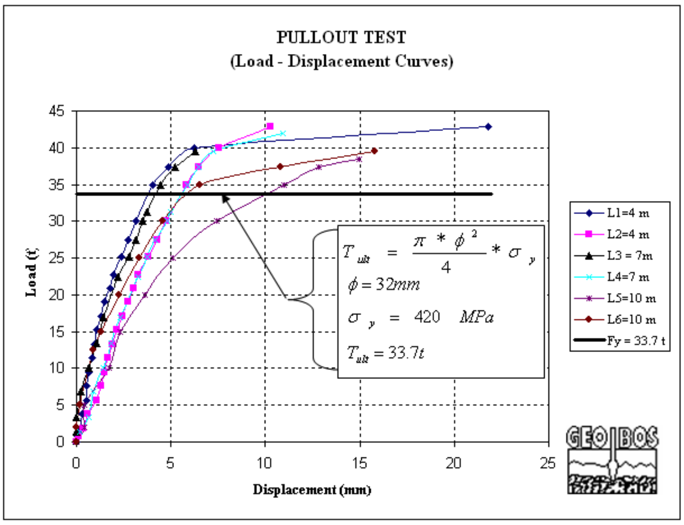

Figure 4. Pullout Test Results (Load – Displacement Curve)

1. All of the nails have been failed by yielding of the rebar. This is apparent by the fact that the ultimate load capacity of a diameter 32 mm rebar, for a yield strength of $\sigma_y$ = 420 MPa, is $T_{ult}$ = 33.7 ton, similar to the average failure load of 38 ton. The 4 t discrepancy is most likely due to the yield strength discrepancy, which has been verified by the rebar manufacturers.
2. For the nail lengths used in this project (minimum L = 4 m), the ultimate capacity of the nail ($T_{ult}$), which is reached at the yield strength ($\sigma_y$), is found to be unrelated to the nail length.
3. For the nail lengths used in this project (minimum L = 4 m), the soil nail does not fail from the grout-soil interface (or the grout-rebar interface). Therefore, no specific skin-friction resistance value can be gathered from the test results.
4. The critical internal failure mechanism for this project is the yielding of the rebar.

From the result of the pullout tests, for the service life of a permanent soil nail wall in greywacke formations, the reduction in the rebar cross-section due to corrosion becomes a very critical parameter in design.

4 DETERMINATION OF SOIL REINFORCEMENT THICKNESS AT THE END OF SERVICE LIFE

In long term (permanent) soil nailing projects, several methods can be employed for protection of passive inclusions against corrosion. These are; plastic or metal encapsulation, galvanizing of steel reinforcement, plastic coating of steel reinforcement.
Another method is to accept the effect of corrosion referred as ‘’Sacrificial Thickness Method’’ in this paper. In this system the steel inclusion corrodes over time and the amount of corrosion is estimated using several approaches. This amount is then converted to a thickness which is sacrificed when determining the axial capacity of reinforcement.  Researchers like Romanoff (1957), Elias

(1990), Anderson (1996) studied corrosion in detail and developed some correlations. Below, these correlations are presented and soil reinforcement thickness at the end of service life for the above project has been determined for each method.

4.1 Romanoff Method (1957)

Romanoff (1957) developed a method to predict the amount of corrosion in metals buried in soil and came up with the following equation.

$$\Delta a = A * t^r \qquad (1)$$

$\Delta a$ = loss of metal thickness/radius ($\mu$m)
A = constant ($\mu$m)
t = buried time (year)
r = constant

Service life (buried time, t) for this project has been planned as 70 years. A non aggressive soil environment has been chosen and constants of A and r has been taken as A = 40 and r = 0.8 “Kentucky Transportation Center (2005)”. Based on this, the sacrificial thickness has been estimated to be $\Delta$a=1.2 mm. Where $\Delta$a is the loss of radius. The reinforcement diameter at the beginning of the project is 32 mm. The effective diameter at the end of project life has been calculated as $\phi_{eff}$= 32 -2* 1.2 = 29.6 mm.

4.2 Earth Reinforcement Method (1996)

In this method, above referenced equation developed by Romanoff is used, but a constant depended on the shape of reinforcement is added. Smith et al suggest that the sacrificed thickness is not the same for flat strips and round bars. Corrosion not only results in the reduction of thickness, but for rounded bars, additional losses are also caused by local pitting “Smith et al (1996)”. In addition to the diameter reduction ($\Delta$a), semi circular local pitting (Sp) is assumed to be effective at one location at the cross section (Figure 5).

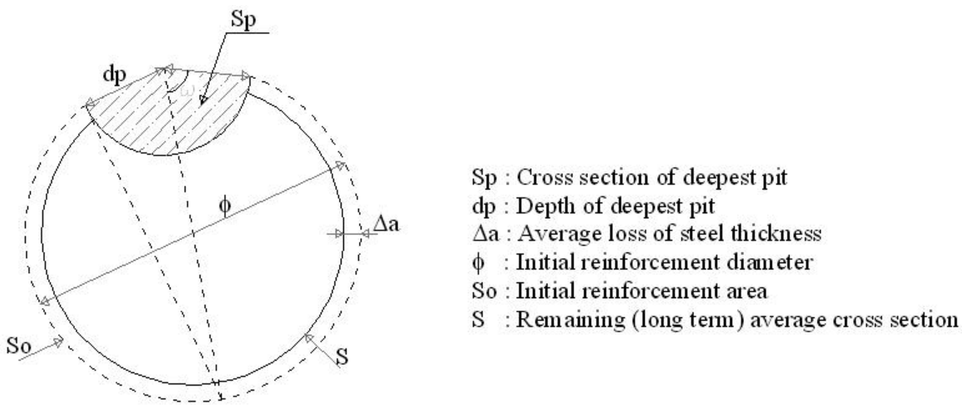


Figure 5. Corroded rounded bar cross section “Smith et al (1996)”

Due to stress concentration at pitting location, relative loss of strength ratio ($\Delta$T/T) is assumed to be more then relative loss of cross sectional ratio ($\Delta$S/S) “Smith et al (1996)”. A shape factor K, which relates loss of cross sectional area to the loss of tensile strength is presented .

$$\frac{\Delta T}{T} = K * \frac{\Delta S}{S} \tag{2}$$

K factor depends on the cross section shape of reinforcement. Studies by Reinforced Earth suggest that rectangular shaped metal strips have smaller K values than rounded bars (Segrestin et al., 1996). At the end of service life, the sacrificial thickness for rectangular shaped reinforcements is less than for rounded bar reinforcements. Another result of this study is that K factor decreases with increasing diameter of rounded bars. For a reduction of $\Delta a = 0.2$ mm in thickness, K factor is determined to be K = 1.78 for metal strip (50 mm x 4 mm), K = 3.11 for diameter 6 mm rounded bar and K = 2.56 for diameter 10 mm rounded bar "Anderson et al (1996)". Shape factor K can be determined using the equations shown below.

$$\Delta S = \pi * (\phi - \Delta a) * \Delta a \tag{3}$$

$$\lambda = \frac{1.1}{\Delta a} \tag{4}$$

$$K = 1 + \frac{\lambda * Sp}{\Delta S} \tag{5}$$

Shape factor K has been calculated as K = 1.87 for the diameter 32 mm rounded bar which was planned to be used in the project. Using the shape factor K and average loss of cross section ($\Delta S$), effective cross sectional area ($S_{ec}$) can be calculated as shown below.

$$S_{ec} = So - K * \Delta S \tag{6}$$

Using equation 7 below, effective diameter reduction ($E_s$) has been determined as 4.7 mm.

$$E_s = \phi - \sqrt{\frac{4 * S_{ec}}{\pi}} \tag{7}$$

The effective diameter at the end of service life has been be calculated as $\phi_{eff}$ =32 - 4.7 = 27.3 mm.

### 4.3 Clouterre Method (1991)

In this method, parameters such as Type of Soil, Moisture Content, pH and Resistivity are used to evaluate the corrosiveness of soils. Table-1, which was originally compiled for buried metal culverts, gives the sum of corrosiveness index (A) of the soil "Clouterre (1991)". According to the laboratory results, the pH level of the greywacke formation for the referenced project is approximately 6, greywacke can be classified as sand -clay, resistivity p > 5000 and the moisture content is less than 20 %., resulting in a global index (**Σ**A) of 4.

Table 1. Corrosiveness Index (Clouterre, 1991)

| Criterion | Features | Weight A of Criterion |
|---|---|---|
| Type of Soil | Sand - Clay | 2 |
| Resistivity (p, ohm/cm) | 5000 < p | 0 |
| Moisture (w%) | w < 20 | 0 |
| pH | 5 – 6 | 2 |
| **Global Index (ΣA)** | | **4** |

The importance of the structure is classified as an index C which is added to the sum of A to determine the overall index I ($I_{overall} = \Sigma A + C$) "Clouterre (1991)". An index C = 0 has been chosen based on the characteristics of the project (no traffic surcharge, no buildings nearby etc.).

A table that relates the overall index I to the service life of the structure is presented in Clouterre. For a service life of 70 years and index $I_{overall} = 4$, a total reduction of 4 mm in diameter has been chosen for the referenced project. The effective diameter at the end of service life has been determined as $\phi_{eff} = 32 - 4 = 28$ mm.

5 ALLOWABLE TENSILE CAPACITY ($T_{all}$) OF SOIL NAILS AND SLOPE STABILITY ANALYSIS

The allowable tensile load at the end of service life for metallic soil reinforcements can be determined using various codes and standards such as French Standard Association (AFNOR), British Standard (BS) and American Association of State Highway and Transportation Officials (AASHTO). The definition of factor of safety and allowable tensile load differ for these standards and codes "Anderson et al (1996)". In our case, AASHTO and Federal Highway Administration (FHWA) specifications are employed and a reduction factor of RF = 0.55 to the yield strength is used for metallic soil reinforcement "AASHTO (1996)". The allowable tensile capacity ($T_{all}$) can be calculated as shown:

$$T_{all} = RF * \sigma_y * \pi * \frac{\phi_{eff}^{\ 2}}{4} \tag{8}$$

Using the above equation, the allowable tensile load of a 32 mm diameter soil nail at the end of service life has been determined for different methods in Table -2.

Table 2. Allowable Tensile Loads

| Method | $\phi_{eff}$,Effective Diameter (mm) | $T_{all}$ (kN) |
|---|---|---|
| Romanoff, 1957 | 29.6 | 159 |
| Earth Reinforcement, 1996 | 27.3 | 135 |
| Clouterre, 1991 | 28.0 | 142 |

The allowable tensile loads for three different approaches has been compared and a value of $T_{all}$ = 140 kN has been accepted. A slope stability analysis for a typical cross section of the referenced project is shown in Figure 6. In this analysis, soil inclusions have been presented as continuous horizontal sheets with $T_{all}$ =140 kN and long term engineering properties (c', $\phi$') have been employed. As shown, the long term factor of safety for this section has been calculated as FS = 1.50 for static condition. FS = 1.10 has been calculated for seismic condition.

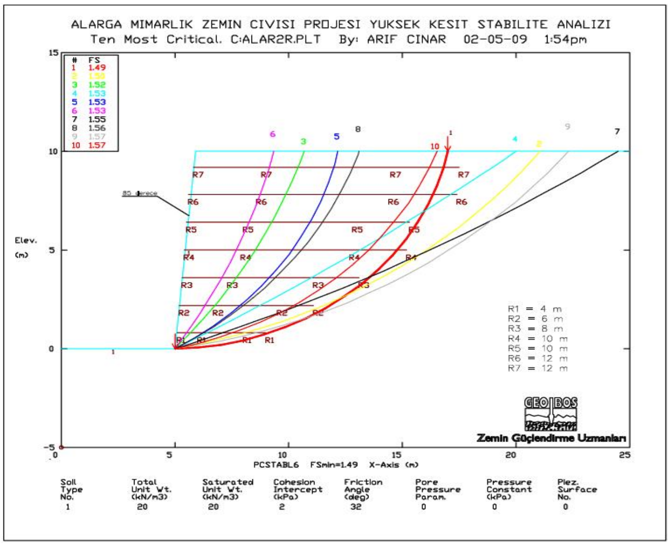


Figure 6. Slope Stability Analysis Results (Stable v.6).

Based on these design parameters a total of 7000 m soil nailing with center to center horizontal distance of 1.5 m and vertical distance of 1.4 m were constructed. The shotcrete area for the project was approximately 1700 $m^2$ (Figure 7).

Figure 7. General View of Soil Nailing Area.

## 6 CONCLUSION AND REMARKS

Pull out tests in greywacke formation have been performed in order to determine the critical internal failure mechanism of soil nails. Nails of various length have been pulled out until failure. It has been concluded that failure occurs not by internal sliding, but by yielding or rupture of the reinforcement. Hence, determining the axial tension capacity of the reinforcement at the end of service life is an important parameter in design. Three approaches have been studied to investigate the sacrificial thickness against corrosion. Based on this study, an allowable tensile load of $T_{all}$ = 140 kN has been accepted for a 32 mm diameter soil nail at the end of service life for this particular project. Factor of Safety obtained from slope stability analysis for the critical cross section has been computed as FS = 1.5 which has been regarded as satisfactory for the project. It has been noted that type of soil, resistivity, pH value, moisture content and the importance of the structure are key players in determining the corrosion and therefore allowable tensile capacity of soil reinforcement. The long term tensile capacity has to be evaluated for each project individually. Drainage is also an important parameter in design. Soil Nailing Walls are not generally designed for hydrostatic pressures. Horizontal drains also act as a precaution against corrosion. Therefore, horizontal drains have to be included in the design.

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