Indian Geotech J (December 2019) 49(6):603–619
https://doi.org/10.1007/s40098-018-0343-1

ORIGINAL PAPER

# Study and Remedy of Kotropi Landslide in Himachal Pradesh, India

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Received: 11 April 2018 / Accepted: 7 December 2018 / Published online: 17 December 2018


**Abstract** A debris flow type of landslide is believed to have propagated from existing minor landslides and heavy rainfall on August 13, 2017, near the village of Kotropi (Mandi District, Himachal Pradesh), India. The disastrous landslide swept away two state transport buses causing 47 fatalities. A stretch of 300-m on National Highway-154 was completely buried under debris by a massive 1153 m of slope run-out extending over 190 m of slope width. The present research work aims at mitigation of Kotropi slope failure using helical soil nails. The preliminary study involves geotechnical and chemical testing of Kotropi soil. With favorable prevailing soil conditions, helical soil nails with length of 6 m and diameter 20 mm are used for stabilizing the failed slope. The stability of helical soil-nailed slope is determined by calculating factor of safety using limit equilibrium method which is also validated by numerical modeling using finite element subroutine PLAXIS 2D. A factor of safety of 1.54 is achieved by calculations in comparison with 1.67 from numerical modeling. Moreover, a decrease in maximum horizontal slope deformation is also achieved from 0.13 to 0.06 m.

**Keywords** Landslide · Helical soil nail · Factor of safety · Finite element · PLAXIS 2D

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## Introduction

Landslide along hilly region is a real problem to engineers and society. The landslides can be man-made and natural. Almost every year they affect the habitat of every creature terribly, leading to massive loss of life and property. During the monsoon season in India, hilly regions of Himachal Pradesh face problem of landslides every year. Urbanization of the region has led to significant climatic and topographical changes. Consequently, excessive rainfall occurs in the region, thus creating a large number of landslide prone zones. On August 13, 2017, a massive landslide occurred in one such zone near the village of Kotropi in Mandi District of Himachal Pradesh, India. The landslide occurred on National Highway (NH)-154, running between Mandi and Pathankot. A section of the slope collapsed completely causing two buses of Himachal Road Transport Corporation (HRTC) along with few other vehicles to be buried under the debris. The vehicles were swept 800 m down the slope by this slope failure. Around 300 m of highway was completely buried under debris, thereby disrupting the communication of the region with adjacent areas. The disastrous debris flow caused casualties of 46 people as reported by the media [1, 2].

The reports [1, 2] also claimed that slope failure was triggered due to excessive infiltration resulting from continuous rainfall in Kotropi region. The present study aims at providing a remedial measure for Kotropi slope using a slope stabilization technique known as soil nailing. Among the various available techniques for slope restoration such as retaining wall, conventional soil nail, rock bolting, anchors [3], soil nailing has proved to be an effective successfully solution for landslide mitigation [4–8]. However, instead of using conventional soil nailing technique, helical soil nails are employed for Kotropi landslide mitigation.

The implementation of conventional grouted soil nails is found to significantly affect the in situ soil properties due to drilling and grouting during nail installation. Conventional soil nails are also difficult to install in soil containing sand, gravels, cobbles and boulders. At such types of ground conditions, drilling and grouting causes large amount of disturbance to the surrounding soil region [9, 10]. To overcome such problem, helical soil nails are used, in which nails are installed directly into the ground by driving the nail with torque action. This allows the nails to be installed without grouting, thereby making it more economical, and also the rotating action facilitates nail penetration without causing significant disturbance to the surrounding soil [3].

The progressive slope failure as in case of landslides generates nonlinear stress–strain conditions within the failing slope. To accurately comprehend the deformation behavior, use of finite element method (FEM) has been recommended by many researchers [4–7]. The geometry of Kotropi landslide is simulated in PLAXIS 2D, a FE-based code. In PLAXIS 2D, soil nail walls/slopes are generally modeled as plane strain problem [8, 9]. The finite element (FE) code predicts the long-term behavior of reinforced structures [10]. Many researchers [11–14] have used PLAXIS 2D for comprehensive study of soil nail structure related to factor of safety (FOS), stress generation during failure, nail forces, wall/slope deformation and failure surfaces.

In this research work, helical soil nails are used to restore the failed Kotropi slope. The helical soil nails typically consists of a circular shaft with small diameter helical flights spaced evenly along the shaft length. Helical soil nail shaft serves a twofold purpose of transmitting axial and torsional stresses to the helical bearing plates, the latter of which is only needed during installation [13]. A typical helical soil nail configuration is shown in Fig. 1.

This paper describes the assessment of both original (unreinforced) and reinforced Kotropi landslide slope by investigating the factor of safety and slope deformation. The factor of safety is calculated using limit equilibrium method (LEM) and finite element method (FEM) using PLAXIS 2D. The results from the LEM calculations are also validated by FEM analysis so as to assess the feasibility of rectified soil-nailed Kotropi slope.

## Study Area

The present study investigates a landslide which occurred near the village of Kotropi, in Mandi District of Himachal Pradesh, India (Fig. 2), which is 414 km from New Delhi and 150 km from the capital Shimla. This place is only 90 km from Dharamshala, which is the wettest place in Himachal Pradesh. The Kotropi region is extended between 31.9121° N latitude and 76.8879° E longitude. Geologically, the area is in a thrusted contact between Siwaliks and Shali group of rocks containing mainly of dolomites, brick red shale, micaceous sandstones, purple clay and mudstones [16]. Since these rocks are weak in strength, when subjected to displacement by thrust, they make this area highly prone to landslides.

## Landslide Classification

Landslides can be classified into various types such as rock compound slide, silt flow slide, clay rotational slide, clay flow slide, earth flow, sand flow, debris flow, mud flow. Figure 3 shows that before the actual landslide, Kotropi region had been suffering local landslide scars at the slope crest. The group of these small and old landslides caused occurrence of large landslide in the area [16]. As per the report [16], Kotropi landslide was a 'debris flow'-type landslide in which the 'debris flow' occurs along with floods comprised of large amount of soil mass flowing in a steep channel. During intense flooding in this steep channel, the stream bed damages the slope, causing massive movement of sediment. The flow usually initiates with a slide, debris avalanche or rock fall. During Kotropi landslide, the channel created by debris flow is about 1155 m from landslide crown.

As the soil mass begins to flow under the debris-type landslide, change in volume of failing slope is restricted due to the movement of soil mass occurring within confined boundaries such as that in a steep channel [17]. Since the movement does not allow for volume change, pore pressure builds up even in coarse-grained soils, thereby leading to liquefaction of soil mass. This leads to a decrease in soil shear strength which makes the slope unstable [18]. Moreover, as the flow moves downstream, the slope bed is weakened by erosion which adds up large amount of debris in the flow [19].

## Geotechnical Investigation of Kotropi Soil

The investigation of geotechnical properties of Kotropi landslide soil is important so as to identify feasibility of soil for helical soil nailing. The length and breadth of landslide are 1153 m and 300 m, respectively. As reported by PH and PP state unit, Chandigarh [20], failure zone for Kotropi landslide, is found to lie between 5 and 8 m. The samples are collected up to a depth of 6 m; however, physical characterization of soil reveals minimal variation

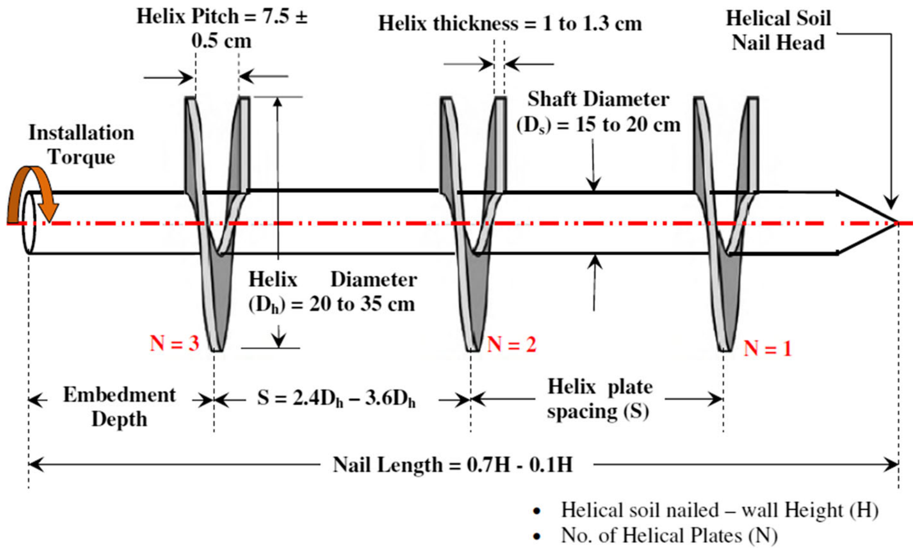



**Fig. 1** Configuration of a helical soil nail. Modified after FSI, [3] and Sharma et al. [15]

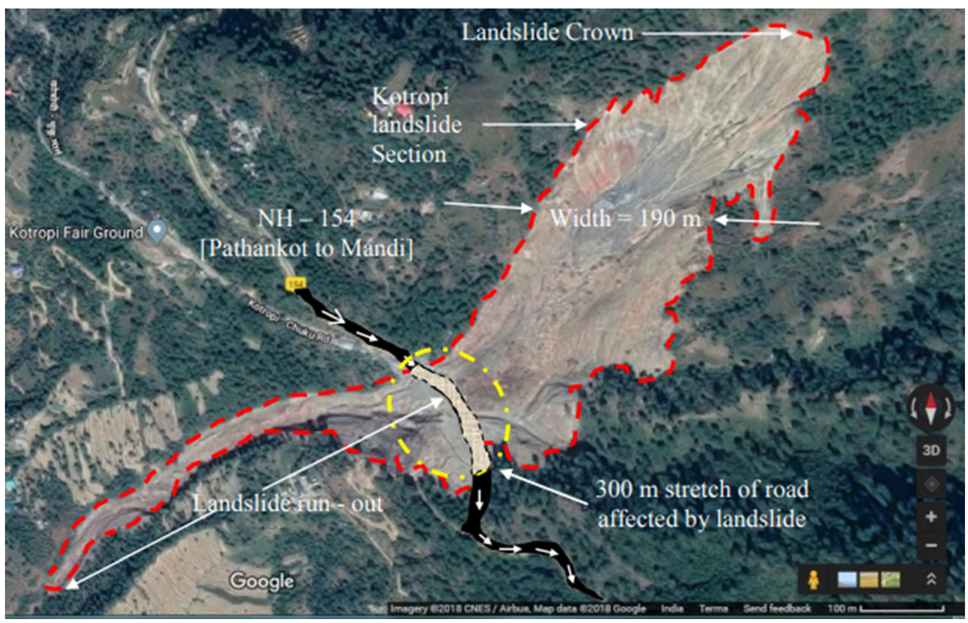



**Fig. 2** Kotropi landslide section

beyond 1.5 m, and hence, results up to 1.5 m depth are only reported. In order to take samples from the site, landslide was equally divided into three sections (upper section, middle section and lower section) along the landslide slope. Each section upper, middle and lower is further divided into three sections 80 m apart to cover the maximum horizontal profile of landslide slope. Thus, the entire Kotropi slope is divided into 9 sections, i.e., 3 (horizontal) and 3 (vertical) from where soil sampling is carried out (Fig. 4). The soil samples from each section are collected using core cutter method in open pits at different depths of 0.5 m, 1 m and 1.5 m. A total of 27 disturbed soil samples are collected, sealed in plastic bags and were transported to Geotechnical Engineering Laboratory at Jaypee University of Information Technology, Waknaghat, Solan, Himachal Pradesh, India, for its characterization. The sampling procedure carried out is in accordance with IS: code 14680-1999 [21].

For characterization of soil samples grain size analysis, Atterberg's limit, compaction test, direct shear test, triaxial shear test and chemical analysis are conducted. The results of these parameters are used for determining the feasibility of helical soil nailing at Kotropi and also for modeling in FE analysis. The grain size analysis is carried out using sieve analysis and hydrometer analysis on all three sections (i.e., top, middle and bottom) of Kotropi landslide at 1.5 m depth (Fig. 5) as per IS: 2720, Part-4 [22]. The tests results depict $C_u$ = 9.30 and $C_c$ = 0.24 for top section, for middle section soil value of $C_u$ = 8.33 and $C_c$ = 0.925 and for bottom section soil value of $C_u$ = 8.31 and $C_c$ = 0.68. The

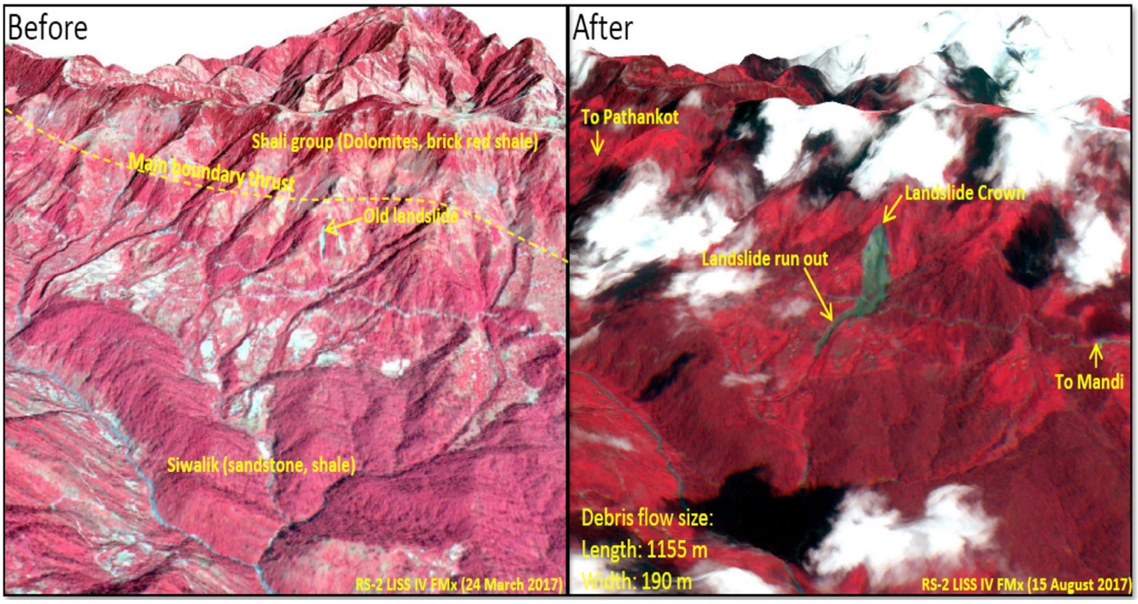
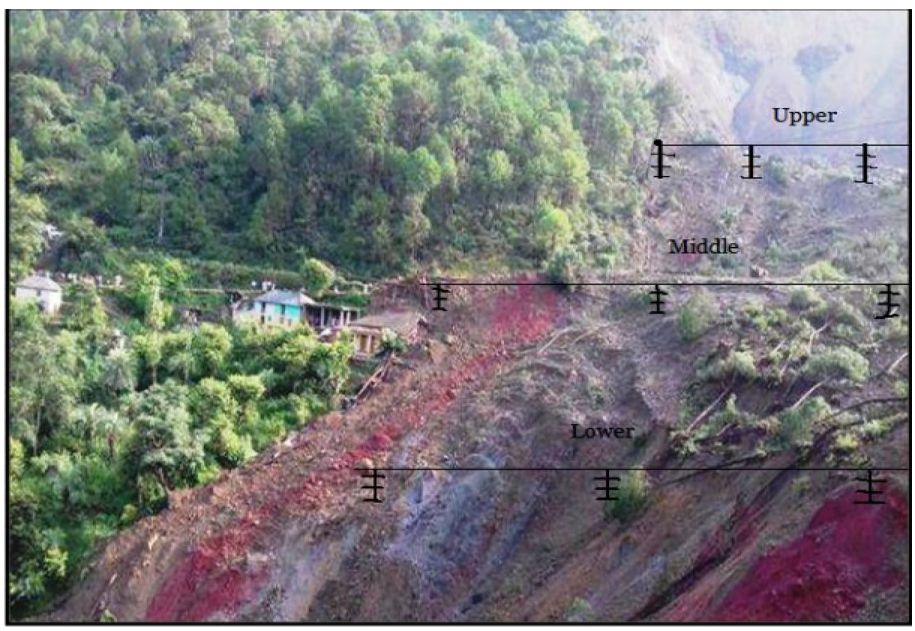



**Fig. 3** Before and after landslide image of Kotropi landslide [16]

**Fig. 4** Sampling point at Kotropi landslide (Mandi, Himachal Pradesh)




particle size distribution also revealed the fineness modulus between 5 and 12%, and hence, the soil is classified as SP-SM (i.e., poorly graded sand containing silt).

However, in order to check the feasibility of helical soil nailing for creep condition, determination of Atterberg's limit is required [14]. Creep tends to induce deformation of soil-nailed structures [14]. Atterberg's limit tests are carried out on three different sections (i.e., top, middle and bottom) of landslide at 1.5 m depth as per IS: 2720, Part-5 [23] (Fig. 6). The results of Atterberg's limit are summarized in Table 1.

The determination of dry density is done by light compaction tests performed as per IS: 2720, Part-7 [24]. Figure 7 represents variation of dry density and water content for different sections of soil at 1.5 m depth. It is found that the soil samples attain a maximum dry density of 1.69 g/cc at an optimum moisture content of 10%.

During 'debris flow' landslide, soil bed is subjected to rapid impact loading condition which results in significant increase in pore water pressure within the failing soil mass [17, 18]. The rapid impact loading is analogous to a short-term loading condition, and geotechnical investigation of Kotropi landslide soil reveals the presence of poorly graded

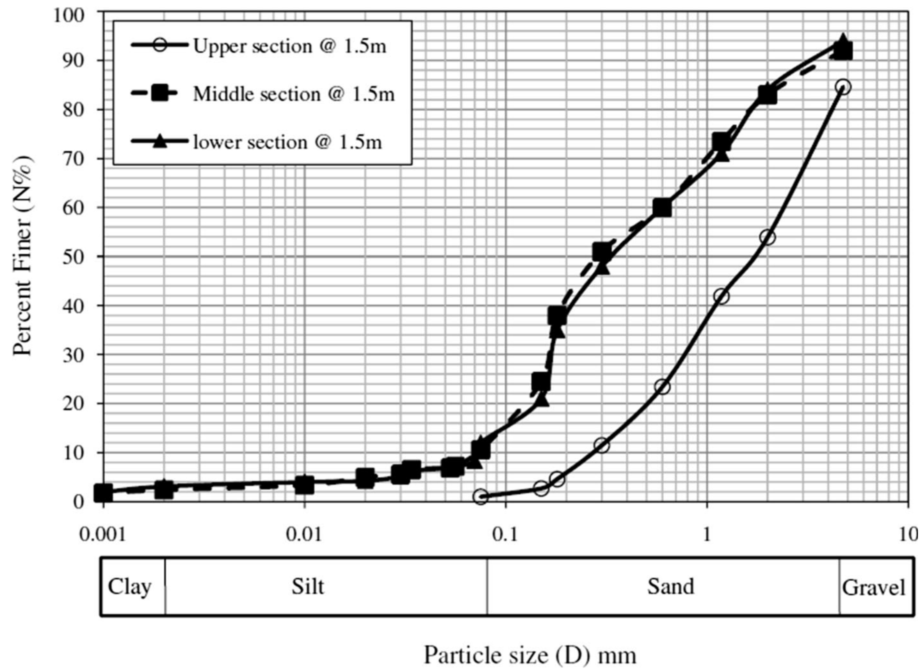



**Fig. 5** Particle size distribution curve

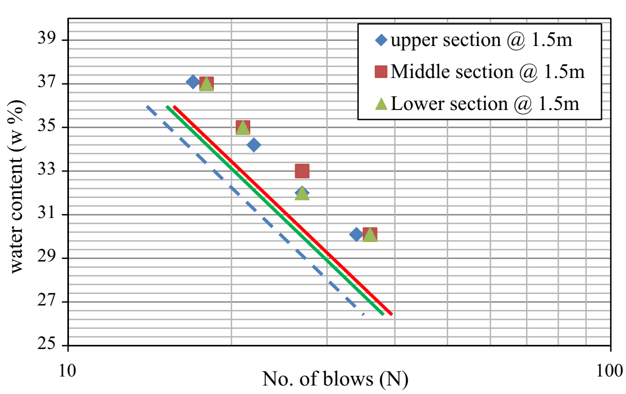



**Fig. 6** Liquid limit of Kotropi soil

**Table 1** Atterberg's limit test results

| Parameter | Upper | Middle | Lower |
|---|---|---|---|
| Liquid limit ($W_L$)% | 32 | 33 | 32 |
| Plastic limit ($W_p$)% | 19 | 16.6 | 16.3 |
| Plasticity index ($I_p$) | 13 | 16.4 | 15.7 |

sand (SP). In such condition, both drained or undrained and total or effective stresses are same, and hence, either of them can be considered to assess the shear strength parameters of the landslide. Based on this knowledge and keeping in mind the presence of small fraction of available silt in the sampled soil, unconsolidated undrained (UU) test is employed for determination of shear strength parameters ($c_u$ and $\phi_u$). It has been found from the literature [19] that UU test has been suggested for soil characterization in cases of debris-type landslide. The drained analysis of soil samples has also been conducted using direct shear test, but since drained and undrained shear strength parameters for sand are equal, only undrained parameters are reported.

The tests are conducted under unconsolidated undrained condition at cell pressures of 50 kPa, 100 kPa, 200 kPa as per IS: 2720, part-11 [25]. From Table 2, it is can be seen that the average value of $c$ and $\phi$ is 26.66 kN/m$^2$ and 32.66°, respectively. The value of cohesion '$c$ = 27.16 kN/m$^2$ can be attributed to the fact that though Kotropi soil mainly consisted of poorly graded sand (SP), apparent cohesion has developed due to the presence of moisture from the infiltration experienced by the slope. Thus, it can be stated that cohesion value so obtained from triaxial testing in fact reveals the apparent cohesion existing between the soil particles. Moreover, the presence of fines in the form of silt content (SM) has also contributed in development of the cohesion value.

## Chemical Properties of Kotropi Soil

According to FHWA [14] and Hubbell helical nail manual [26], in situ soil conditions are required to be checked for safety of soil nails against corrosion and creep for serviceability condition. Hence, in order to check the long-term serviceability of helical soil nails, chemical characterization through pH, chloride and sulfate content of soil is necessary.

Helical soil nailing is not recommended for acidic soil (pH value less than 5) which contains high level of soluble

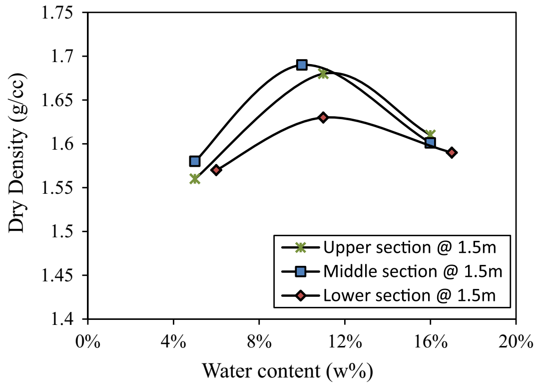



**Fig. 7** Compaction curves for top, middle and lower sections of slope

**Table 2** Shear Strength parameter for all three sections of landslide by using UU triaxial test

| Landslide section | $c$ (kN/m$^2$) | $\Phi$ |
|---|---|---|
| Top | 26 | 32.5° |
| Middle | 26 | 33° |
| Bottom | 28 | 32.5° |

iron, thereby increasing the corrosion potential. Moreover, soil basic in nature (pH value is greater than 7) is also not suggested to be suitable as it may contain sodium, calcium and calcium magnesium carbonates which are mildly corrosive. In the present study, pH value of Kotropi soil is found between 6.5 and 7 which signifies the feasibility of helical soil nail with respect to pH.

In addition to pH, soil containing more than 200 ppm of sulfate and 100 ppm of chloride is also categorized as aggressive soils [14] with the view that such soil promotes corrosion of steel at relatively fast rates. Hence, sulfate and chloride contents of Kotropi soil are also determined to check the threshold value for non-aggressive soil which implies that the level of corrosion can be tolerated with reasonable confidence. The tests are conducted as per AASHTO290 [27] and AASHTOT291 [28].

From the test results (Table 3), it is observed that sulfate and chloride contents in Kotropi soil are within permissible limits [14]. According to FHWA [14], if sulfates and chlorides are within permissible limits, then only galvanization of soil nails is required without any specific pretreatment of soil.

## Feasibility of Helical Soil Nails at Kotropi Landslide

The Kotropi soil is classified as poorly graded sand containing silt. The percentage of chloride and sulfates is within the permissible limit, and the nails are free from corrosive action of chemical like chloride and sulfates. The obtained test results are compared with favorable soil conditions for soil nailing as shown in Table 4, which exhibit the feasibility of helical soil nails at Kotropi landslide.

## Advantages of Helical Soil Nail Over Conventional Soil Nail

Helical soil nails are beneficial over conventional nail as they provide the opportunity of easy installation without significant soil disturbance and spoil production. The helical plate facilitates ease of penetration by application of torque. Moreover, helical soil nails do not require grouting for establishing interface bond between grout-nail and grout-surrounding soil. The required interaction is provided by the bearing from helical plates and interface friction between shaft and surrounding soil. Thus, using helical soil nails not only reduces the requirement of grout material but also makes installation process economical and quicker. These nails are passive bearing elements, which play the role in movement of soil mass and active earth pressures to mobilize soil shear strength along the nail.

## Theoretical Factor of Safety of Helical Soil-Nailed Slope

Theoretical factor of safety is used to determine soil-nailed wall stability which includes geometry problem, soil properties and nail tension. The analysis is based upon limit equilibrium method (LEM), which presents basic principles for safe design of constructed or natural earth slopes. A detailed sketch of helical soil-nailed slope depicting various forces acting on slope sections, nail location, probable slip surface and corresponding soil properties is shown in Fig. 8. The factor of safety using the

**Table 3** Chemical properties of collected soil samples

| Properties | Top section | Middle section | Bottom section | Recommendation (as per FHWA [18]) |
|---|---|---|---|---|
| Chloride content (mg/L) | 60 | 40 | 80 | < 100 |
| Sulfate content (mg/L) | 66.6 | 133.3 | 190 | < 200 |
| Soil pH | 6.5 | 6.6 | 6.5 | 5–10 |

**Table 4** Comparison of obtained test results with reference manual for favorable condition for soil nailing

| Properties | Ground condition | Soil creep potential | Soil corrosion potential |
|---|---|---|---|
| Parameter for the present work | For top section soil<br>$C_u$ = 9.3 and $C_c$ = 0.24<br>For middle section soil<br>$C_u$ = 8.33 and $C_c$ = 0.925<br>For bottom section soil<br>$C_u$ = 8.31 and $C_c$ = 0.68<br>It is clear that from $C_U$ and $C_C$ values the soil can be classified as SP-SM (i.e., poorly graded sand containing silt) | For top section<br>Liquid limit, $W_L$ = 32<br>Plastic limit, $W_p$ = 19<br>Plasticity Index = 13<br>For middle section<br>Liquid limit, $W_L$ = 33<br>Plastic limit, $W_P$ = 16.6<br>Plasticity Index, $I_P$ = 16.4<br>For bottom section<br>Liquid limit, $W_L$ = 32<br>Plastic limit, $W_P$ = 16.3<br>Plasticity Index, $I_P$ = 15.7 | pH = 6.5 (all three sections)<br>Conc. of sulfates (mg/L)<br>Top = 66.6 (mg/L)<br>Middle = 133.3 (mg/L)<br>Bottom = 190 (mg/L)<br>Chloride content (mg/L)<br>Top = 60 (mg/L)<br>Middle = 40 (mg/L)<br>Bottom = 80 (mg/L) |
| Remark (as per FHWA [14] and Hubbell helical nail manual [26]) | (1) Soil nailing is favorable for dense to very dense granular soil with apparent cohesion, weathered rock with adverse weakness planes, stiff to hard fine-grained soils residual soil and glacial fill<br>(2) Favorable for poorly graded, cohesion less soil $C_u > 2$ | (1) If liquid limit ≥ 50 and plasticity index ≥ 20, then it is considered that creep may occur in soil, which is not favorable for soil nailing<br>(2) Soil creep is deformation of the wall resulting reduction of the shear strength of the soil. Therefore, liquid limit < 50% and plasticity index < 20 are favorable for soil nailing because soil does not meet the criteria for creep potential | (1) pH should lie between 5 and 10<br>(2) Sulfate content should be less than 200 (mg/L)<br>(3) Chloride content should be less than 100 (mg/L) |

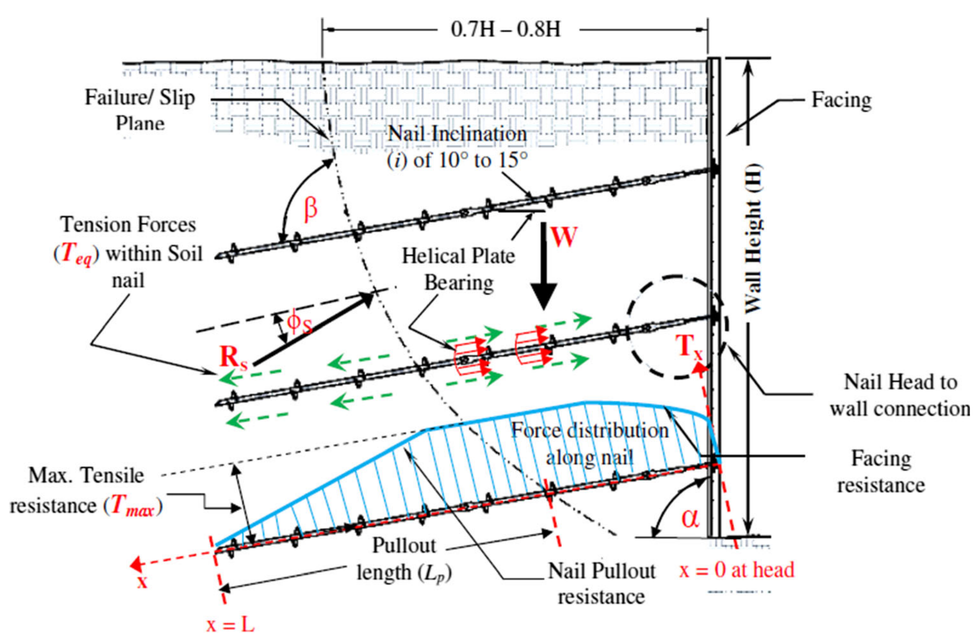



**Fig. 8** Various forces acting in a helical soil-nailed wall. Modified after FSI [3] and FHWA [14]

force equilibrium of different soil wedges as adopted from FHWA [14] is obtained from Eq. (1).

$$\mathrm{FS} = \frac{(T_{\mathrm{eq}}\cos(\Psi - i) + [(W+Q)\cos\Psi + T_{\mathrm{eq}}\sin(\Psi - i)]\tan\phi)}{(W+Q)\mathrm{Sin}\Psi} \quad (1)$$

where

$$T_{\mathrm{eq}} = \text{equivalent nail force} = \sum_{j=1}^{n}(T_{\mathrm{all}}) \quad (2)$$

$$W = \text{weight of the failure wedge} = 0.5\gamma H^2 \cot^{\varphi} = 439\,\mathrm{kN/m} \quad (3)$$

$$K_{\mathrm{a}} = \frac{1-\sin\phi}{1+\sin\phi} = 0.301 \quad (4)$$

The average maximum tensile force in the upper two-thirds of the wall,

$$T_{\mathrm{eq}} = \sum \mathrm{Tall}(U) + \sum \mathrm{Tall}(L) \quad (5)$$

$$T_{\mathrm{all}_{(u)}} = 0.75K_{\mathrm{a}}\gamma_{\mathrm{s}}HS_{\mathrm{v}}S_{\mathrm{H}} = 0.75 \times 0.301 \times 16 \times 10 \times 1 \times 1 = 36.12\,\mathrm{kN} \quad (6)$$

Upper 2/3 of the 10-m-high wall contains 7 nails.

$$\sum T_{\mathrm{all}_{(u)}} = 7 \times 36.12 = 253\,\mathrm{kN} \quad (7)$$

$$\text{Maximum tensile forces in the lower one - third of the wall} = 0.55K_a\gamma_s HS_v S_H \quad (8)$$

$$T_{\mathrm{all}_{(\mathrm{L})}} = 0.55 \times 0.301 \times 16 \times 10 \times 1 \times 1 = 26.48\,\mathrm{kN} \quad (9)$$

Lower one-third of the wall contains 3 nails

$$\sum T_{\mathrm{all}_{(L)}} = 3 \times 26.48 = 79.46\,\mathrm{kN} \quad (10)$$

$$T_{\mathrm{eq}} = 79.46 + 253 = 332.46\,\mathrm{kN} \quad (11)$$

$$\Psi = 45^\circ + \frac{\Phi}{2} = 61.25^\circ \quad (12)$$

$i$ = nail inclination of soil nail wall with horizontal = 15° (Adopted from Rawat and Gupta [29]).

Therefore, stability safety factor (FOS) = 1.54 ≥ 1.35.

Factor of safety against sliding according to Hubble [26]

$$K_{\mathrm{a}} = \tan^2\left(45 - \frac{\Phi}{2}\right) \quad (13)$$

$$K_{\mathrm{a}} = \tan^2\left(45 - \frac{32.5}{2}\right) = 0.30 \quad (14)$$

The horizontal force from the retained soil is determined using Eq. (15) as:

$$F = \frac{1}{2}K_{\mathrm{a}}\gamma H^2 = \frac{1}{2}(0.3) \times (16) \times (10^2) = 240.71\,\mathrm{kN/m} \quad (15)$$

Helical soil nails are installed at 15° angle, adopted length of nail = 0.6H

$$\text{Factor of safety against sliding is determined as follows: } \frac{\gamma \mathrm{HL}\tan\phi}{F} \quad (16)$$

Factor of safety = 2.54; which is ≥ 1.5. Hence, it is safe.

## Numerical Modeling Using Finite Element Method

### Geometrical Definitions of the Model

Simulation of the actual site condition has been carried out by finite element method (FEM) using PLAXIS 2D. From the length of 1155 m of landslide, only 60 m of slope height (i.e. 30 m above and 30 m below from National Highway-154) is repaired such that the road section can be constructed and made open to use. The entire height of Kotropi slope is divided into vertical segments of 10 m each. The soil is removed from top 10 m so as to improve the stability of constructed segments. However, FE analysis with top 10 m intact with the slope has also been carried out to check the variation in FOS for restored helical soil-nailed Kotropi slope. With the removal of top 10 m of slope, the effective slope height is 20 m above the road (NH-154) as shown in Fig. 9.

As per IS: code 14680:1999 [21], procedure of benching is required for achieving stability of slopes. The procedure involves dividing the long slope into smaller segments. The geometry of each segment is determined by error trial such that each helical soil-nailed section is stable against failure with FOS greater than 1.5. In order to achieve this, some sections have been assigned vertical slope. Moreover, vertical slope also facilitates easy helical soil nail installation. Table 5 summaries the geometric configuration and other design details of the helical soil nail wall.

### Material Properties

The Kotropi soil is modeled using Mohr–Coulomb (MC) model. MC model is an elasto-plastic model, which combines Hooke's law and the Coulomb's failure criterion. The present helical soil-nailed slope design is primarily based on deformation. As reported in the literature [30] for progressive slope failure model to investigate the strain-softening behavior, elasto-plastic analysis is required. Moreover, large displacement reinforced slope problems are best evaluated using elasto-plastic analysis. Therefore,

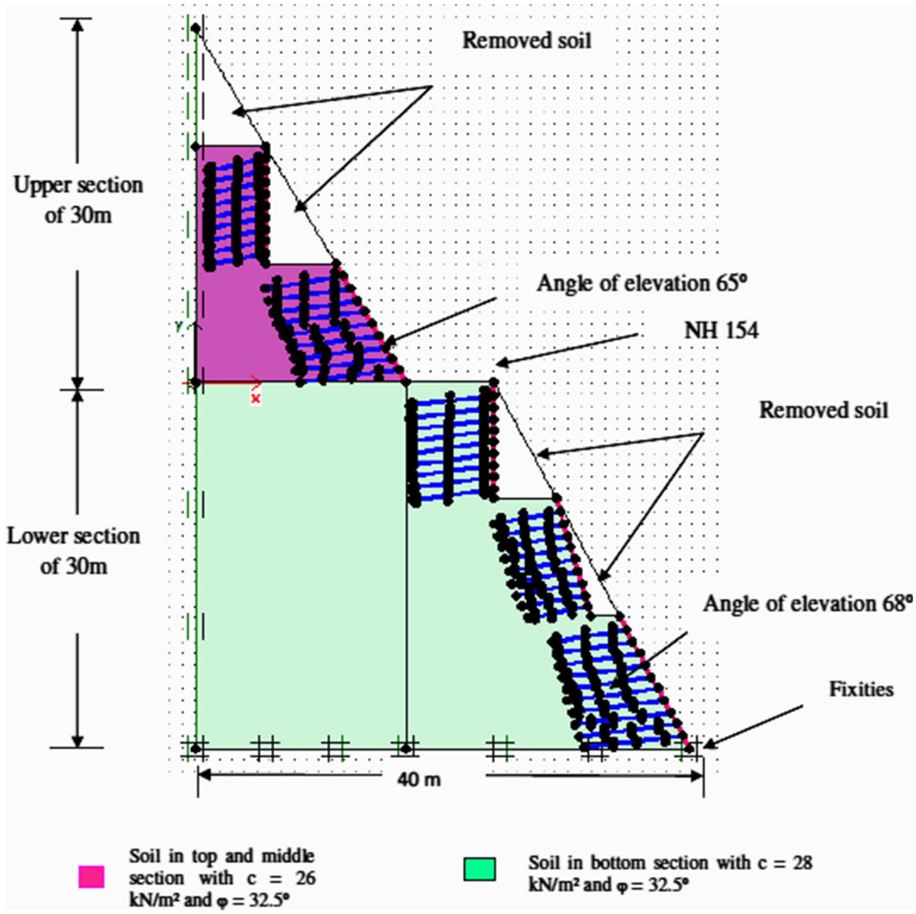



**Fig. 9** Geometrical configuration of finite element model of Kotropi slope

**Table 5** Helical nails wall geometry and other parameters

| Parameters | Parameters adopted in the design |
|---|---|
| Nail length | 6 m |
| Vertical height of the wall | 30 m |
| Vertical height of each segment | 10 m |
| Nail type | Helical nail (without grout) |
| Nail inclination | 15° |
| Nail spacing ($S_h \times S_v$) | 1 m × 1 m |
| Elasticity modulus of reinforcement ($E_n$) | 200 (GPa) |
| Thickness of facing | 225 mm |
| Slope angle | 65° for upper section and 68° for lower section |
| Unit weight $\gamma$ (kN/m$^3$) | 16 |
| Diameter of helical nail | 20 mm |

helical soil-nailed Kotropi slope is simulated as an elasto-plastic model to overcome the shortcomings involved in factor of safety prediction of slopes involving large displacement through limit equilibrium method (LEM).

The depth and subsoil properties employed for modeling original Kotropi slope in PLAXIS 2D are adopted from the geotechnical investigation carried out on the soil samples collected from the area under study. Care is taken that soil sampling is conducted beyond the failure zone so that the characteristics of original slope are incorporated into the FE analysis. However, among the determined $C_u$ and $\phi_u$ values at various depths, the minimum values are adopted in FE analysis for worst-case scenario. The various soil model parameters adopted are listed in Table 6.

The entire problem is modeled in plane strain condition and for long-term condition using drained analysis. The prevalent soil conditions at Kotropi landslide found after geotechnical investigation depicted poorly graded sand containing silt (SP-SM) soil. During debris flow at Kotropi, the in situ coarse-grained soil is assumed not to have led to generation of pore water pressure even under rapid impact loading condition. Since the shear strength parameters for undrained ($c_u$ and $\phi_u$) from UU test and drained ($c'$ and $\phi'$) from CD test conditions for coarse-grained soil are similar, UU test shear strength parameters can be used for assessing long-term behavior of the slopes also. Consequently, total and effective stresses are also equal for coarse-grained soils, since SP-SM soil will not support generation of any

**Table 6** Helical nail modeling parameters in PLAXIS 2D

| Parameters | Values (units) |
|---|---|
| Helical soil nail | |
| Modeling element | Plate |
| Modeling type | Elasto-plastic |
| Modulus of elasticity of helical nails ($E_n$) | 200 GPa |
| Equivalent modulus of elasticity ($E_{eq}$) | 200 GPa |
| Equivalent axial stiffness (EA) | $0.06280 \times 10^{-3}$ kN/m |
| Equivalent bending stiffness (EI) | $2.093 \times 10^{-9}$ kN-m$^2$/m |
| Equivalent plate diameter ($d_{eq}$) | 20 mm |
| Kotropi slope soil | |
| Cohesion ($c$) | |
| Upper slope section | 26 kN/m$^2$ |
| Middle slope section | 26 kN/m$^2$ |
| Lower slope section | 28 kN/m$^2$ |
| Angle of friction ($\phi$) | |
| Upper slope section | 32.5° |
| Middle slope section | 33° |
| Lower slope section | 32.5° |
| Modulus of elasticity of soil ($E_{soil}$) | $9.6 \times 10^3$ kN/m$^2$ |
| Poisson ratio of soil ($\mu$) | 0.3 |
| Dilatancy angle of soil ($\psi$) | 0 |

pore water pressure during failure. Thus, $c_u$ and $\phi_u$ values determined through UU test have been used for drained analysis for investigating the long-term behavior of helical soil-nailed Kotropi slope.

For modeling helical soil nails, plate elements are used [8, 9]. The material parameters used for structural elements simulating soil nails are the axial stiffness *EA* and flexural rigidity *EI*. For helical soil nails, an equivalent modulus of elasticity (*Eeq*) is also determined for accounting the contribution of elastic stiffness of reinforcement bar. As per Babu and Singh [9], equivalent modulus of elasticity (*Eeq*) is calculated from Eq. (17) as:

$$E_{eq} = E_n\left(\frac{A_n}{A}\right) + E_g\left(\frac{A_g}{A}\right) \tag{17}$$

where $E_n$ is the modulus of elasticity.

$A_n$ is cross-sectional area of helical nail, $A$ is gross area of nail, $A_g$ is cross-sectional area of grouted soil nail, $E_g$ is modulus of elasticity of grout material, and $E_n$ is the modulus of elasticity of nail. Since no grouting is done during helical soil nail installation, $A_g = E_g = 0$. Moreover, the cross-sectional area of nail ($A_n$) and gross area of nail ($A$) will also be equal.

$$\therefore E_{eq} = E_n\left(\frac{A_n}{A}\right) \tag{18}$$

where $A = 0.25\pi D_n^2$ is the total cross-sectional area of soil nail. If $S_h$ is horizontal, $S_v$ is vertical spacing of soil nails and $D_n$ = diameter of helical nail, then axial and bending stiffness [9] can be obtained by Eqs. (19) and (20) as:

Axial stiffness (kN/m)

$$\mathrm{EA} = \frac{E_n}{S_h}\left(\frac{\pi D_n^2}{4}\right) \tag{19}$$

where ‘n’ subscript indicates nail

Bending stiffness (kNm$^2$/m)

$$\mathrm{EI} = \frac{E_n\left(\pi d_n^4\right)}{S_h 64} \tag{20}$$

Since the helical soil nails have circular shaft as adopted for the present design, plate elements are converted to circular section with equivalent plate diameter of nail using Eq. (21) as:

$$d_{eq} = \sqrt{12\frac{\mathrm{EI}}{\mathrm{EA}}} \tag{21}$$

## Numerical Analysis of Helical Soil-Nailed Slope

Once the material properties of soil and helical soil nails are defined, boundary conditions are modeled using standard fixities available in PLAXIS 2D package. The base of slope is fixed in $x$–$y$ direction with the back of the slope being restricted only in the x-direction. The slope face is free to move in both x and y directions, respectively. The top of the slope is also free to move in vertical direction [31].

The modeling of soil nail interface is done by using a strength reduction factor ($R_{inter}$) value. To assure appropriate soil–nail interaction, an interface of virtual thickness factor ($\Delta$ = 0.1) is used. This factor ($\Delta$) is multiplied by the thickness of element in mesh generation procedure. The interface is allotted similar properties to that of corresponding soil section. As per Brinkgreve [32], strength reduction factor ($R_{inter}$) is used to model the interface friction between nail and soil during failure. The $R_{inter}$ refers to shear strength parameters of soil with joint strength as:

$$R_{inter} = \frac{\tan\phi_{interface}}{\tan\phi_{soil}} \tag{22}$$

$$R_{inter} = \frac{c_{interface}}{c_{soil}} \tag{23}$$

As can been seen from Eq. (22), $R_{inter}$ value models the interface friction that will be mobilized between soil and nail in case of granular soil and similarly for fine-grained soil, where cohesion predominates, Eq. (23) is used. The discretization of modeled slope is carried out by using medium size mesh for soil domain with fine meshing in regions around helical nails to accurately model the interaction between soil and nails. To model equilibrium conditions for Kotropi helical soil-nailed slope, initial stress is generated using $K_0$-procedure through Janbu's relation. This procedure simulates the earth pressure at rest condition.

The modeled slope is then analyzed using staged construction for the fact that soil nailing installation is carried out in stages. The 10-m top soil of slope is removed by deactivating its cluster. Since installation of helical soil nails is carried out after every 1 m, excavation depth of 1 m is simulated by deactivating the corresponding soil cluster in every calculation stage. A total of 8 calculations stages are defined for the entire Kotropi slope of 30 m. The reinforced Kotropi slope is also provided with a concrete facing, which is also modeled using plate elements with properties of concrete.

Finally, the helical soil-nailed Kotropi slope is analyzed for safety and plastic deformation. In PLAXIS 2D, safety factor for slopes is determined using strength reduction method [8, 9, 31]. The shear strength parameters of the soil are continuously reduced until slope failure. The strength of plate and anchors is not influenced by Phi/c reduction. A factor known as total multiplier $\sum$Msf is used to define the value of soil strength parameters [32] at a given stage of analysis as given in Eq. (24).

$$\sum \mathrm{Msf} = \frac{\tan \phi_{\mathrm{input}}}{\tan \phi_{\mathrm{reduced}}} = \frac{c_{\mathrm{input}}}{c_{\mathrm{reduced}}} \tag{24}$$

The slope deformation behavior is attained from its plastic analysis. The complete FE model of reinforced Kotropi slope with helical soil nails is shown in Fig. 9.

## Finite Element Results for Factor of Safety

The factor of safety (FOS) calculation yields a value of incremental multiplier $\sum$Msf which is found to become concurrent at failure. According to Brinkgreve et al. [32], value of $\sum$Msf represents the factor of safety, which is plotted against Cartesian displacement ($|U|m$) of slope. However, the Cartesian displacements are not relevant for factor of safety, and it only indicates whether or not a failure mechanism has developed.

The analysis of original unreinforced Kotropi slope reveals that as deformation occurs, the soil tends to detach itself from the slope. The analysis terminates with a result that 'soil body seems to collapse.' This clearly signifies the occurrence of landslide due to transition of soil into its plastic state. A similar observation is also made while locating the plastic zones during failure. It is observed that top of the slope is found to detach itself as it cuts off from the remaining slope under tension. The slip failure occurs along the zone where the soil has moved into plastic deformation. Hence factor of safety for unreinforced Kotropi slope cannot be determined as it fails which reflects a FOS $< 1$. However, after installation of helical soil nails, an increase in factor of safety is obtained. The reinforced Kotropi slope is analyzed for both cases of with and without the top 10 m of soil. It is observed that factor of safety of 1.57 is obtained with top 10 m of soil as shown in Fig. 10.

However, from Fig. 11, it can be observed that the factor of safety is found to increase to 1.67 with the removal of 10 m of slope at the top. The percentage increase in factor of safety is found to be 6.4% with soil removal at top 10 m of slope. Hence, during restoration of slope it is recommended that top 10 m of soil should be removed to achieve a better FOS. It is found that factor of safety obtained after nailing is greater than 1.5 which is the permissible value for global factor of safety of soil-nailed structures [14]. Therefore, it can be stated that the designed helical soil nails can render stability to the Kotropi slope against failure.

## Validation of Factor of Safety

It is observed from both theoretical calculations (LEM) and numerical method (FEM) that factor of safety is higher than overall stability (FS = 1.5). The LEM gives a FOS of 1.54, whereas FOS of 1.67 is achieved from PLAXIS 2D. The difference in LEM factor of safety and factor of safety obtained from FEM may be due to fact that LEM primarily involves equilibrium of forces acting on soil wedge, whereas FEM-based PLAXIS 2D considers elastic–plastic deformation of nodes. The latter is more accurate as it takes into consideration helical soil nail–soil interaction while nails are only considered as stabilizing force in LEM.

## Finite Element Results for Failure Surface

Figure 12 depicts critical slip failure for original unreinforced Kotropi slope corresponding to a factor of safety (FOS) which is found to be less than 1. During the finite element modeling in PLAXIS 2D, the soil body is found to have collapsed reflecting the failure of original slope during landslide (Fig. 15). The top of Kotropi slope is found to have undergone tension cut-off depicted by white zone. The soil lying in this zone is found to have detached itself from the original slope and moved down the slope face in the form of a debris flow. The red zone reveals the regions on the slope where permanent deformation of soil has occurred. This zone is also the probable slip surface during Kotropi landslide. Figure 12 also reveals that tension cut-off points and plastic points lie along similar soil failure zones which further strengthen the discussion over the movement of slope as depicted in Fig. 15.

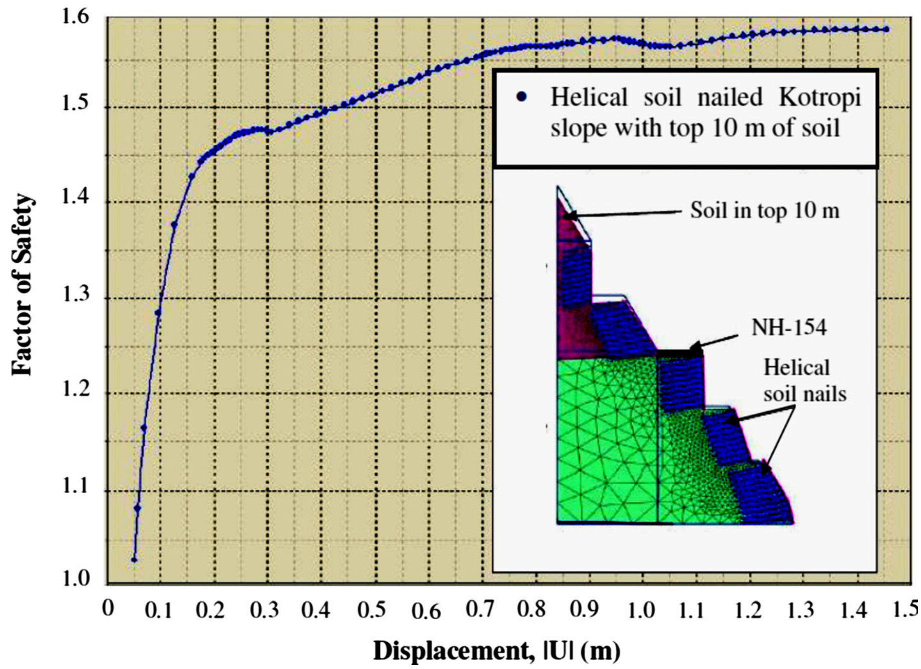



**Fig. 10** Factor of safety for reinforced slope with top 10-m soil

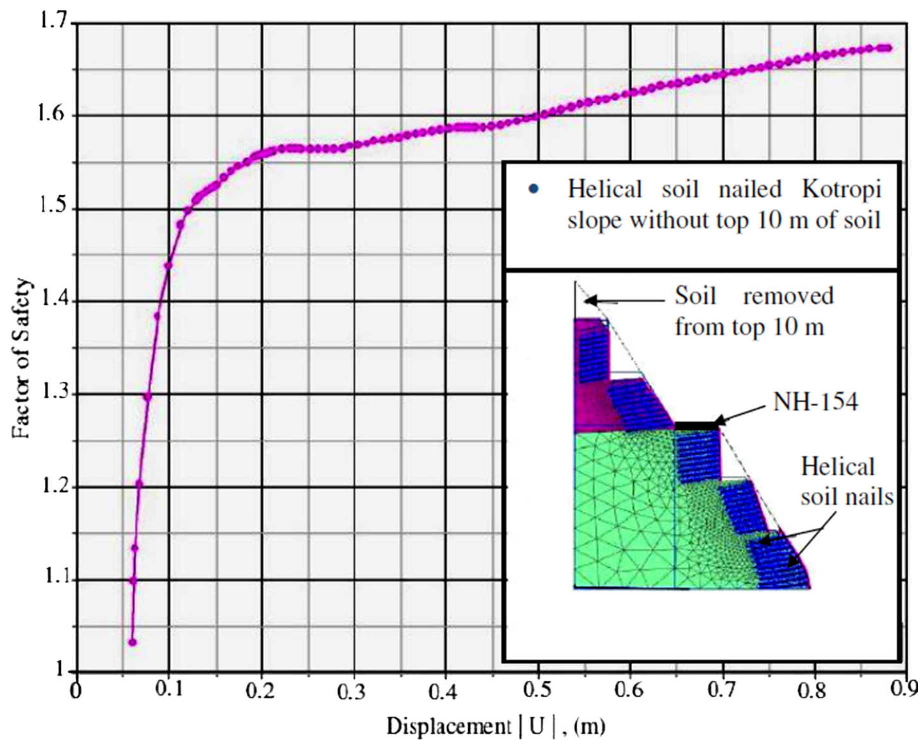
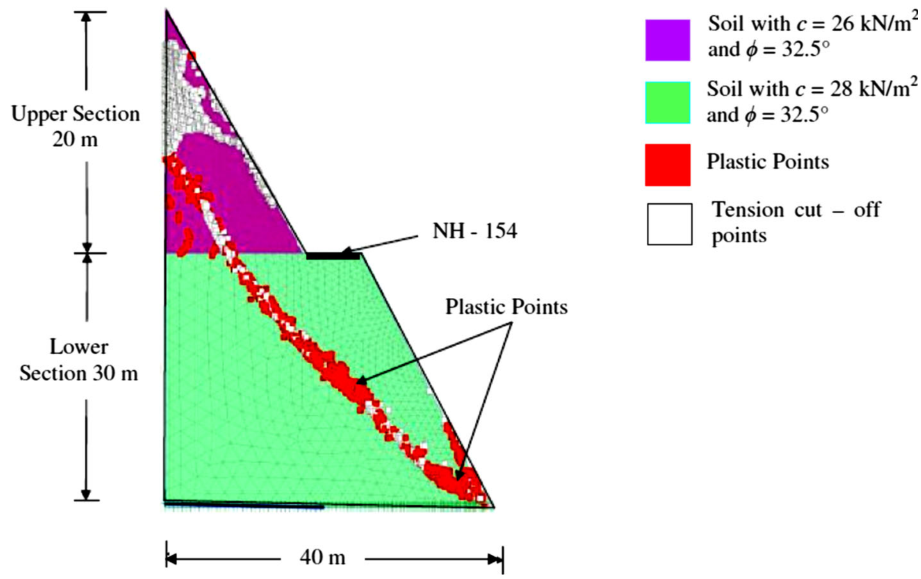



**Fig. 11** Factor of safety for reinforced slope without top 10-m soil

The depth of failure surface during landslide mainly depends upon the properties of soil and its thickness. In case of slope being in homogeneous soil condition, depth of failure surface is the height of slope and the bottom soil is stiffer than top soil. In non-homogeneous soil, it depends purely on the type of soil and its thickness. In the present study, the soil type is found to be homogeneous but anisotropic. From PLAXIS 2D by using distance measurement feature, depth of failure surface was 3 m from the top, 8 m at the center and 0 m at the bottom. Similar results are also reported by GIS team after preliminary assessment of Kotropi landslide [20], where the depth of failure surface was found to be 5–8 m.

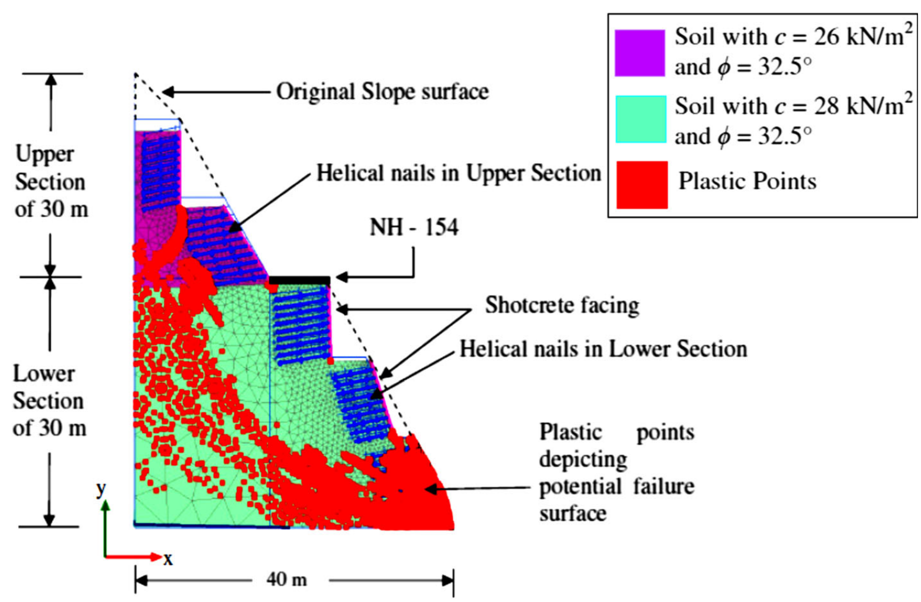
Fig. 12 Plastic point of unreinforced slope




Though the entire slope comprises poorly graded sand, properties are found to be different for slope top, middle and bottom. The critical slip surface is obtained by investigation the failed region which has undergone plastic deformation during slope failure. PLAXIS 2D provides the opportunity to locate these plastic points as shown in Fig. 13. It is also observed from Fig. 13 that the slip surface has moved to a deeper zone with nailing of different slope sections as compared to unreinforced slope. With the transition of failure surface to a deeper zone, shear resistance along the failure surface increases, thereby yielding a factor of safety greater than 1.

The plastic zone for reinforced slope in comparison with unreinforced slope shows the absence of tension cut-off zone. Moreover, no clearly defined slip surface is obtained for reinforced slope as compared to unreinforced slope where plastic points accompanied with tension cut-off points contribute toward development of landslide. Due to interaction of helical soil nails, interface friction increases between soil and nail. Because of the soil deformations, the interface friction increases with respect to time due to increased soil settlement and consequently increased shearing resistance along helical nails. The strains generated around helical soil nails help in resisting the destabilizing force resulting in stabilized slope. The helical soil

Fig. 13 Plastic point of slope after nailing



nails are also found to provide additional resistance due to bearing from helical plates. The diameter of helical plates allows large volume of soil to interact with helical nail, which creates helical soil nails acting as large diameter nails [33].

The stability of a soil-nailed system primarily depends upon its internal stability and general global stability. The internal stability corresponds to stability contribution from nails, whereas general global stability reflects stability with no contribution from nails. The nails are found to contribute toward stabilization through mobilization of its tensile, pullout and facing resistance. Among these, tensile strength and facing resistance are mobilized whether or not the slip surface is intersecting with the nails. If the slip surface is found to intersect with the nails, the pullout resistance is mobilized and contributes toward internal stability [14, 34]. The soil nails in sections A, B, C and D (Fig. 4a) reflect to a similar condition where only the tensile strength of helical nails and facing resistance are found to render stability during slope deformation. Thus, it can be stated that sections A, B, C and D are stabilized by only by tensile resistance and facing resistance of the corresponding helical nails in their respective locations. Moreover, the general global stability is also found to have been achieved as depicted by a FOS $>$ 1.5 [14] for helical soil-nailed Kotropi slope.

## Nail Forces

The nail forces developed in the helical nails are found to be compressive and tensile in nature. As can be seen in Fig. 14a, the top section (i.e., slope above NH-154), all helical nails are found to be under tensile forces. This reflects the fact that reinforcing action due to nail is significantly achieved for the upper portion of the Kotropi helical soil-nailed slope.

However, nail forces in lower portion of rectified slope are found to be both tensile and compressive. The last row of nails in the lower 10 m below the highway is found to depict compressive forces (Fig. 14b). Any stabilization measures like soil nails, rock bolts are found to be effective if they are located in the zones of tensile strains generated during deformation. Thus, location and orientation of nails plays a vital role in the type of forces that will be mobilized during failure. It is also observed that the nail forces tend to undergo transition from tension to compression if the angle between the normal to the slip surface and nail is found to change from positive to negative [11].

As observed from Fig. 14a, b, most of the helical nails are found to act under tension since they do not intersect with the failure surface. This clearly reflects that nail inclination of 15° with horizontal is effective in rendering the reinforcing action to respective sections. However, it can be observed from Fig. 14b, bottom two rows of helical soil nails in section ‘c’ and last four rows of helical soil nails in section ‘d’ depict helical nails under compression. The reason for this variation can be contributed to the fact that for these sections the local failure slip surface must have been terminating at toe of the section, thereby intersecting through the lower rows of helical nails. The orientation of these rows of nails must have changed the angle between slip surface normal and nail inclination from

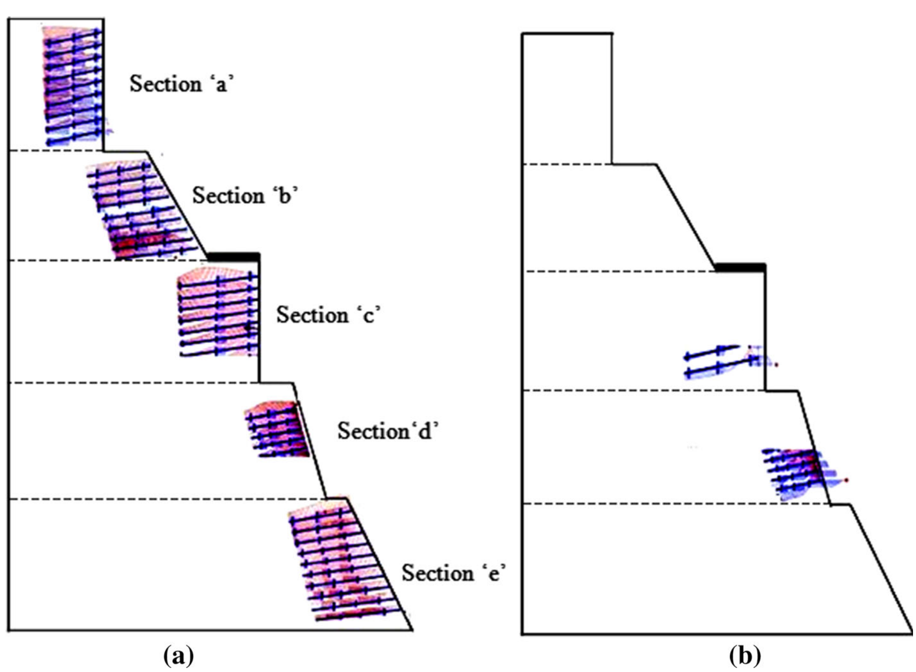



**Fig. 14** **a** Tensile forces in helical nails. **b** Compressive forces in helical nails

**Table 7** Maximum axial forces in section

| Section | Maximum nail force | Nature of force |
|---|---|---|
| A | $0.14 \times 10^{-6}$kN/m | Tension |
| B | $0.40 \times 10^{-6}$ kN/m | Tension |
| C | $0.11 \times 10^{-6}$ kN/m | Tension |
| D | $0.16 \times 10^{-6}$ kN/m | Tension |
| E | **$8.16 \times 10^{-6}$ kN/m** | Tension |

The maximum tensile force in nail is found to be for the lowermost section denoted by 'E'

positive to negative. Thus, making the nails lie in zones of compressive strains instead of tensile strains. Hence, mobilization of compressive forces is found.

The axial forces of helical soil nails are affected due to inclination. Due to increase in nail inclination, reinforcing forces decrease in nails. The force in some nails shifts from tension to compression due to variation of angle between nail inclination and normal to failure surface from positive to negative which makes the nails location close to direction of compressive strain developed during failure instead of tensile strains [11]. Moreover, location of plastic point as given in Fig. 13 shows higher concentration of failure points at toe of slope which can be attributed to mobilization of only tension forces in the last 10 m portion of helical soil-nailed slope.

The maximum force in soil nails was observed to be $8.16 \times 10^{-6}$ kN/m. This force was tensile in nature and observed at the bottommost section (e) of the slope. Maximum axial forces in each section are listed in Table 7.

## Assessment of Lateral Displacement

In Kotropi slope, decrease in shear strength of soil is due to heavy rainfall, which led to displacement in slope [18, 26]. The lateral displacement can be predicted well for unreinforced slope from Fig. 15. It is clear that there is large displacement occurring over the unreinforced slope due to decreases in shear strength of soil. The unreinforced slope is found to have undergone a total deformation of 13 cm predominantly at the crest of the slope.

According to FHWA [14], maximum long-term horizontal displacements at the top of the wall can be estimated for poorly graded sand by Eq. (25)

$$\Delta h = \Delta V = \left(\frac{\Delta h}{H}\right) \times H \tag{25}$$

And also,

Maximum lateral displacement = 0.2% of vertical height [14].

Here, total height of slope = 60 m

$$\frac{\Delta h}{H} = \frac{1}{500}, \quad \text{for } c - \Phi_{\text{soil}} \tag{26}$$

Thus, permissible slope displacement for helical soil-nailed slope as obtained from Eqs. (25) and (26) is found to be 0.12 m. Moreover, FE analysis of rectified helical soil-nailed Kotropi slope shown in Fig. 16 yields maximum displacement of 0.06 m. Thus, stabilization of Kotropi slope using helical nails is found satisfactory for serviceability condition also, i.e., displacement of helical soil-nailed Kotropi slope $< 0.12$ m (permissible limit).

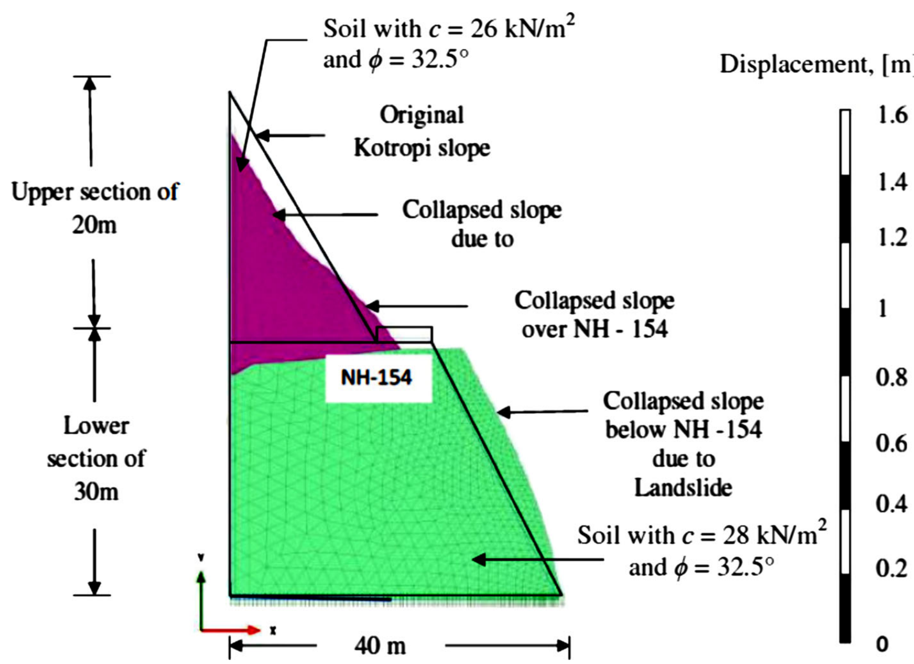



**Fig. 15** Slope deformation of unreinforced Kotropi slope

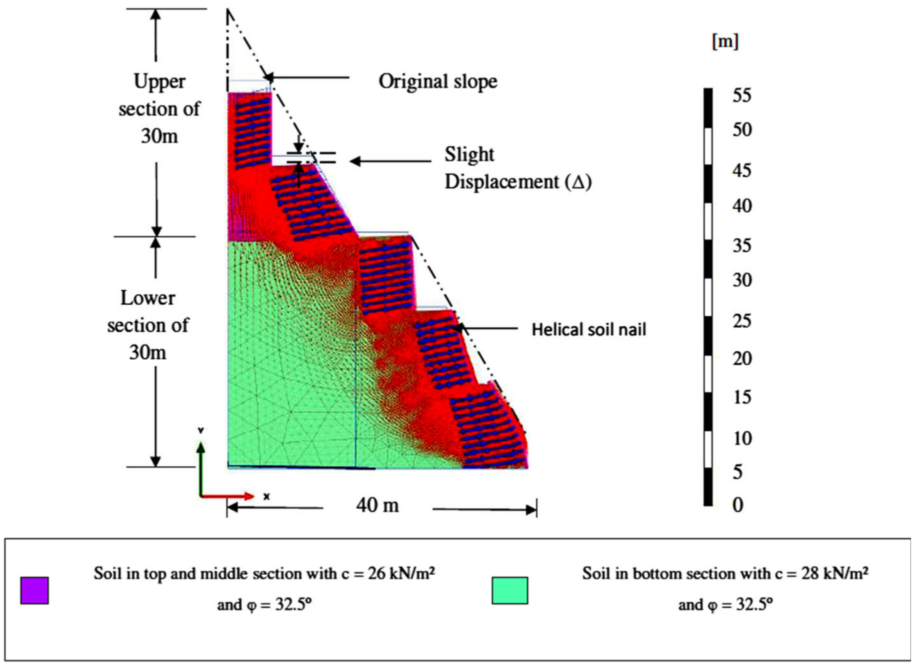



**Fig. 16** Deformation of helical soil-nailed Kotropi slope

Hence, it can be stated that suggested helical soil nail design for restoring slope stability is satisfactory.

## Conclusions

The present research work includes the geotechnical and chemical soil investigation of Kotropi landslide. In addition to evaluation of factor of safety from LEM, the FEM analysis of stabilized Kotropi landslide slope is also carried out using helical soil nails. The factor of safety, deformation and nail forces of unreinforced and reinforced Kotropi slope have been presented and compared. Based on the results obtained, mitigation of Kotropi landslide using helical soil nails is suggested. The following conclusions can be derived from the present study:

1. The Kotropi slope without soil nail is found to collapse reflecting a FOS (factor of safety) < 1. The factor of safety is found to increase to 1.67 by using helical soil nails for restoring the Kotropi slope which is greater than global safety factor 1.5. It can be concluded that slope stabilization can be achieved from the given helical soil nail design.
2. The deformation of original Kotropi slope is found to reduce from 0.13 to 0.06 m for unreinforced and reinforced slopes, respectively. Also, the numerical analysis of helical soil-nailed Kotropi slope depicted that slope displacements are within permissible limits which is conclusive for assessing the feasibility of helical soil nail performance under serviceability condition.
3. From the nail force distribution, it can be concluded that nail forces are found to develop tensile forces which signifies efficient reinforcing action of installed helical nails.

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