Long Term Landslide Mitigation Technique Illustrated- A Case Study

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Abstract

Landslide is the mass movement comprising of rock, debris or soil under gravity influence. The main slide triggering factors are rain and tectonic seismicity induced landslides occurring in the Himalayan belt of the Indian subcontinent. Anthropogenic activities like tunnel blasting for hydroelectric projects, unplanned excavations or cuttings of the side for road widening purpose activates failure mechanism. On the other hand, continuous blasting and tunnelling weaken the rock joint layers. Based severity of rock joints and rock surface conditions; rock structures have been characterised into different range of GSI chart. Thorough study and analysis are required to check the long term stability for these type of weak slopes with GSI range of 10 to 40 having fair, poor to very poor surface conditions lying close to important and sensitive structures. The present study reveals the effectiveness of preventive measure applied to the unstable slope stretch in the vicinity of Teesta stage III hydroelectric system. Finite element modelling has been carried out for the critical slide triggering stretch using Rocscience-Phase2 v8.005. Analysis has been carried out without and with stability measures. It has been observed that 50m vertical cladding wall having prestressed cable anchor with no base support survived the 2012 earthquake of 7 Richter magnitude scale with no signs of distress.

Keywords: Landslide; Himalayan belt; Long term stability; Finite element modelling; Prestressed cable anchor; Teesta stage III hydroelectric system

Introduction

Construction of dams in the seismic prone area is a challenging task. This requires proper geological and topographical analysis in order to evaluate structural behaviour

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and to obtain critical safety factors. Fig. 5.1 depicts factor causing the reduction in the shear strength in the same strata and adjoining layer. It can be seen clearly from this figure that how rain infiltrates through the persistent and non-persistent joints causing a decrease in shear properties and consequently creating detachment of block in blocky rock structure and slide in the highly jointed rock mass. The continuous build of pore water pressure generates a flow of muck or loose mass overlying the water-resisting strata.

The Teesta stage III hydro project is a huge mega-dam in Sikkim comprises of concrete face rock dam of 60 m height constructed across Teesta River near Chungtang village, which has seen multi-faceted impacts on the indigenous Lepcha people of Sikkim.

Fig. 5.1: Landslide mechanism induced due to Rainfall

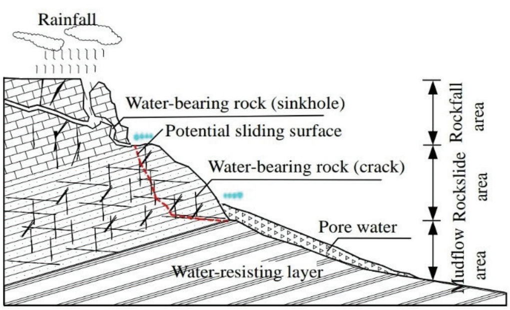


Fig. 5.2: Typical slided zone along road

Fig. 5.3: Map showing Teesta Stage III location

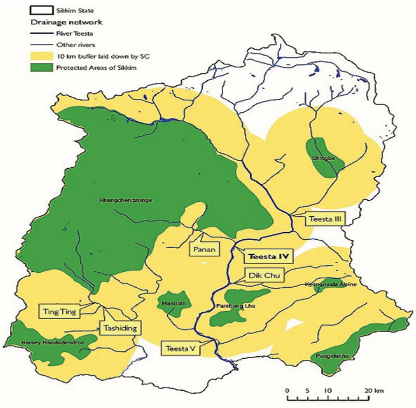


Fig. 5.4: Protection works for unstable portion

The unstable portion was observed during construction activities at the dam site and cutting of left abutment which was susceptible to failure. GSI of 25 shows the rock structure of disintegrated- poorly interlocked, heavily broken rock mass with a mixture of angular and rounded rock pieces. This value of GSI suggests a weak rock condition.

METHODOLOGY

Geological site description

Site geology of dam site area lies in the rock formation of central crystalline; subdivided into Chungthang series, Darjeeling gneiss & Rongli series of Pre-Cambrian age. Chungthang series comprises of quartz, biotite gneiss, biotite schist, calc-silicate and thinner bands of argentiferous – sillimanite heavily micaceous gneissose. Rock units are highly folded; asymmetric, isoclinal in nature with NE dips.

Landslide issue was observed due to excessive stripping and cutting of the left abutment as shown in Fig. 5.5. About 1 lac m^3 debris slide down the hill after the cutting work at the left abutment of the dam site.

Fig. 5.5: Site condition before the start of work

Fig. 5.6: Before commencement of cable anchoring

Geological Strength Index (GSI) of the slope is computed based on rock mass structure and surface condition of discontinuities with reference to GSI chart published by Hoek and Brown (1997). The extent of weathering, joint roughness and infilling material between the discontinuities, joint apertures were studied and analysis; based on which ratings were assigned to reach GSI value (Pandit et al., 2019). GSI value of 25 was observed showing rock structure of disintegrated- poorly interlocked, heavily broken rock mass with a mixture of angular and rounded rock pieces.

Calculating shear strength reduction factors

The finite element of slope requires selection of constitutive relation between stress-strain behaviour, which depends upon the material type. Equivalent continuum modelling with elastic and plastic ranges is adopted for these type of disintegrated rock layers.

In a present case study of FEM analysis, Generalised Hoek Brown (Hoek et al., 2002) with material constants "m_b, "s" and "a" have been reported and derived from GSI values (Equation 1-3) and verified from Rocscience roclab data software (Pandit et al., 2019). GHB material model adopted for the gneissose and surface weathered rock, while Mohr-Coulomb material model for overburden soil.

$$m_b = m_i \cdot \exp\left(\frac{GSI-100}{28-14D}\right) \tag{1}$$

$$s = \exp\left(\frac{GSI-100}{9-3D}\right) \tag{2}$$

$$a = \frac{1}{2}+\frac{1}{6}\left(e^{-GSI/15}-e^{-20/3}\right) \tag{3}$$

The extent of the problem domain of left abutment of Teesta dam site has been modelled, discretized with fine mesh density. The slant height of slope is 226 m having an average inclination angle of 62^0. The average depth of overburden mass and top weathered surface rock is around 24 m, 12 m respectively. The slope has been modelled with boundary conditions having slope face as free, vertical sides as roller support allowing vertical displacement, the base is restrained from any moment (i.e.hinged support). Six noded triangular with uniform type mesh is adopted for the study as recommended in Rocscience manual (2012).

Fig.5.7: Schematic diagram of proposed cable anchor stabilization at left abutment of dam axis

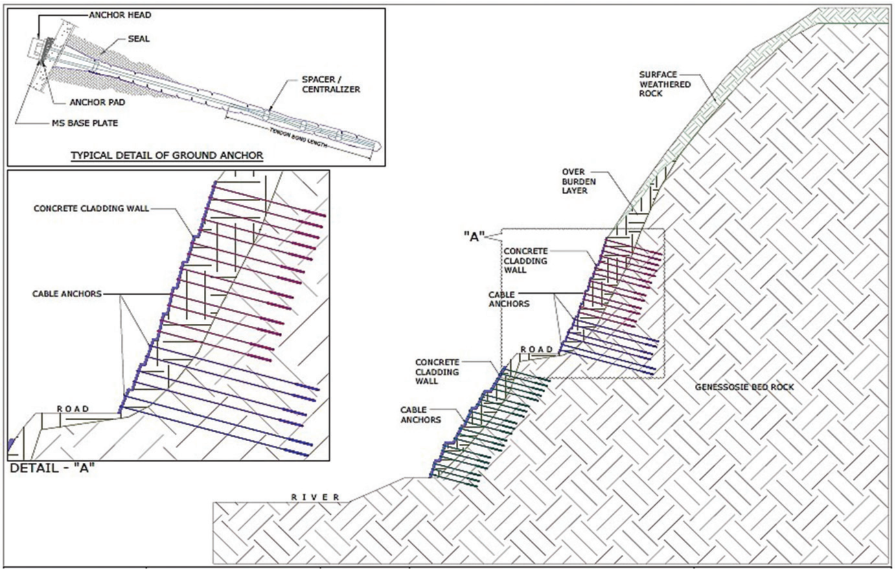
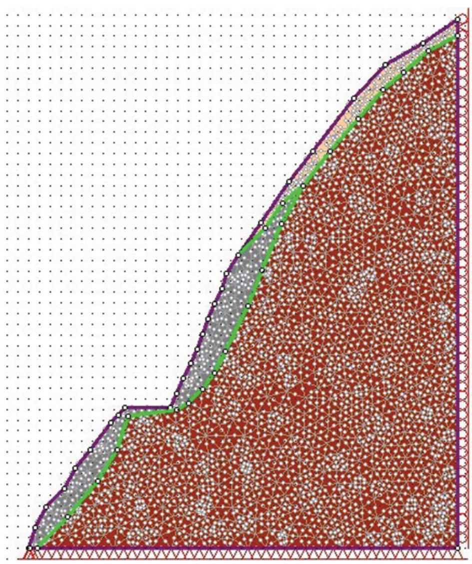
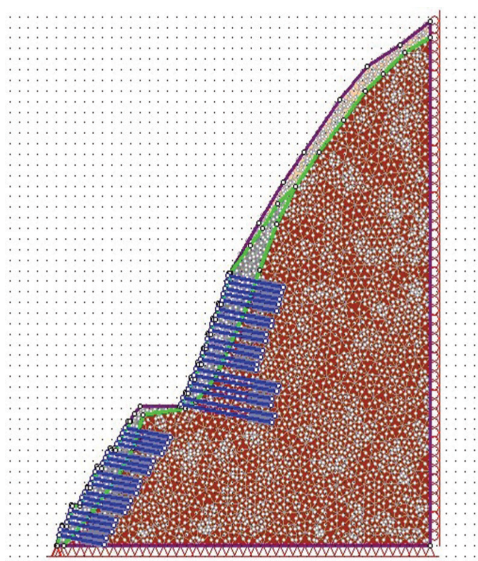


Fig. 5.8: Typical discretised mesh of slope Figure 5.9. Discretised mesh of slope with stabilisation





Once material properties are assigned and mesh discretization is done, the slope stability analysis has been performed for static and seismic analysis (Zone IV) having a coefficient of the horizontal seismic load as 0.15 and coefficient of the vertical seismic load as 0.1 acting vertically downwards and away from slope; which is the severe load combination. Pore water pressure with Ru as 0.25 has been considered for the analysis.

Table 5.1: Properties of Materials used in FEM Analysis

Material Type	Overburden	Surface weathered rock	Bed rock (Genessosie)
Material Model	Mohr Coulomb	Generalized Hoek Brown	Generalized Hoek Brown
Unit weight (kN/m^3)	21	22	24
Cohesion (kN/m^2)	98	215	482
Friction angle (degree)	28	17.88	19.79
mb	-	0.066	0.108
s	-	1.62e-06	3.73e-6
a	-	0.544	0.531
Intact UCS (Mpa)	-	70	120

Once all material properties and discretisation has been done for static and seismic condition, the slope is analysed to assess critical strength reduction factor. Slope models have been also analysed after applying the stabilisation measure in the form of prestressed cable anchor with RCC cladding wall. The cladding wall of 500 mm thickness is lifted in stages of 7 to 10 metres. Taking about the construction method, the slope protection using cable anchors with cladding wall is a top-down construction process, so there is minimal stress occurs at the toe of the slope. For the present case study, the toe protection is applied above and below the slope bench. As per IS 14448-1997, the fixed anchor length is designed based on the three factors such as failure of rock and grout bond, failure of grout/anchor bond and failure of the anchor. Based on these recommendations, failure criteria of rock and gout bond observed as critical with bond stress 160 kN/m and fixed anchor length as 12 m. Perforated drainage pipes are installed along the slope profile (cladding wall) to relieve the pore water pressure.

Table 5.2: Properties of Stabilised Material used for Protection System

Material Type	Prestressed cable anchor
Anchor type	Tie back member
Diameter (mm)	150
Ultimate tensile capacity (tonne)	100
Length (m)	25 to 40 m
Out plane spacing (m)	3
In plane spacing (m)	3
Bond stress (kN/m)	160
Pre tensioning force (tonne)	120

Results

Analysis of left abutment at dam axis shows that the slope is quite unstable under the static and seismic condition with SRF 0.95 and 0.81.

Fig. 5.10: Horizontal displacement (Left side) and Total displacement (Right side) for unstable slope under static condition

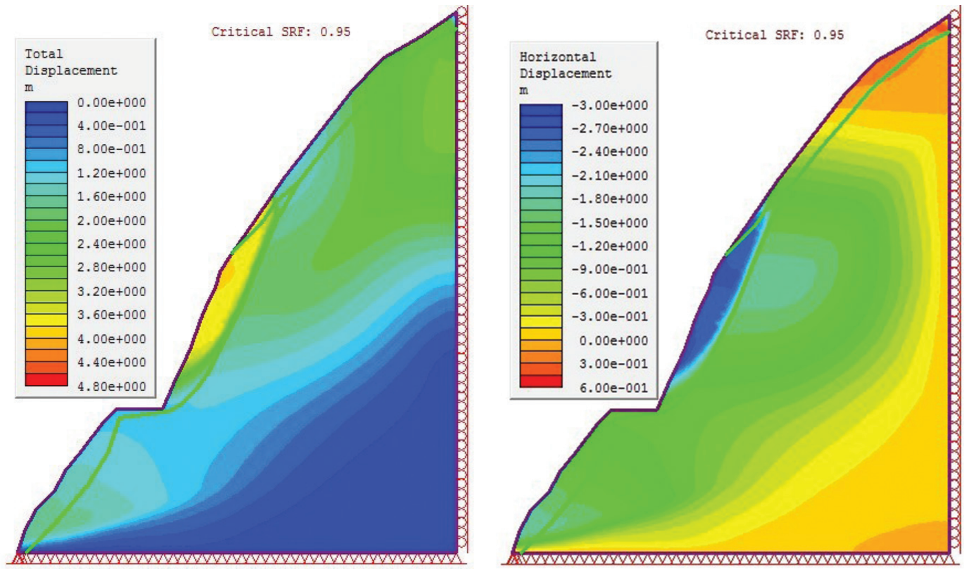
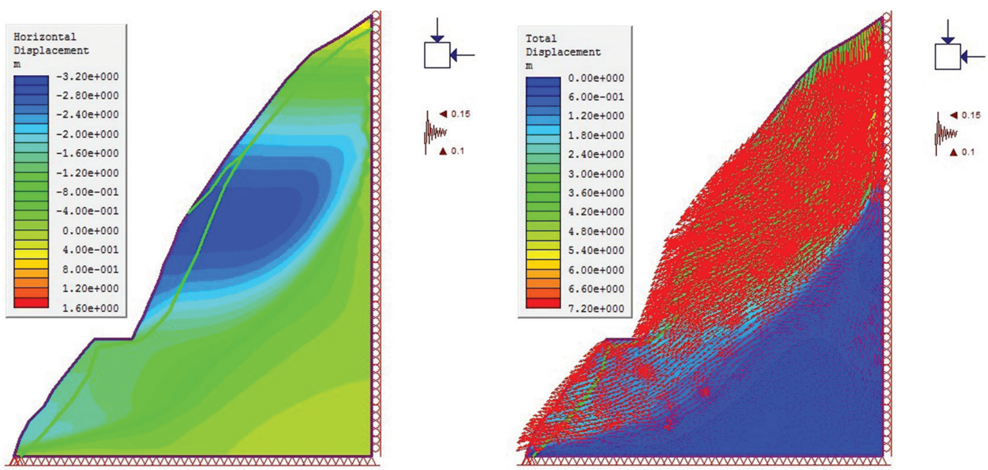


Fig. 5.11: Horizontal displacement (Left side) and Total displacement (Right side) for unstable slope under seismic condition



Following results depicts the effect of inclusion of prestressed cable anchors in reducing horizontal displacement, total displacement. Critical SRF value of 1.65 with 41 mm displacement for the static case (Figs. 5.10 to 5.14) and 1.32 critical SRF with 47 mm displacement has been observed under seismic case (Figs. 5.15 to 5.17).

Fig. 5.12: Horizontal displacement (Left side) and Total displacement (Right side) for stabilised slope under static condition

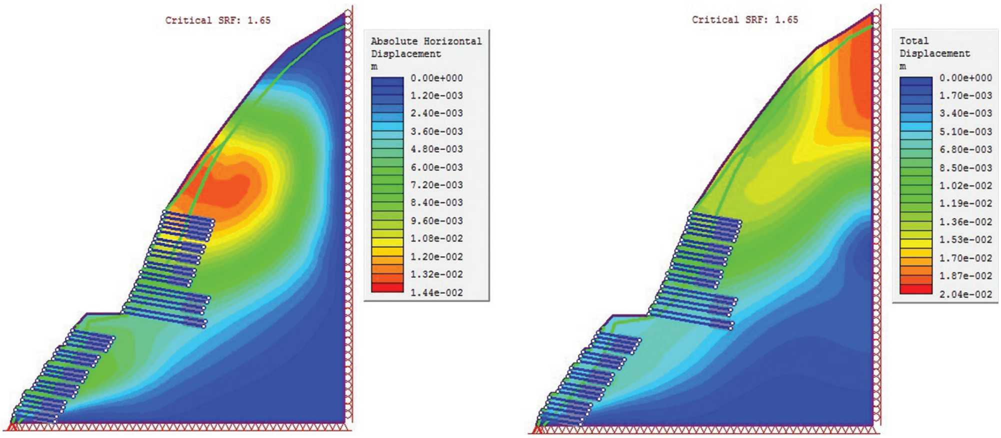


Fig. 5.13: Variation of strength reduction factor with maximum total displacement for static case (with stabilisation)

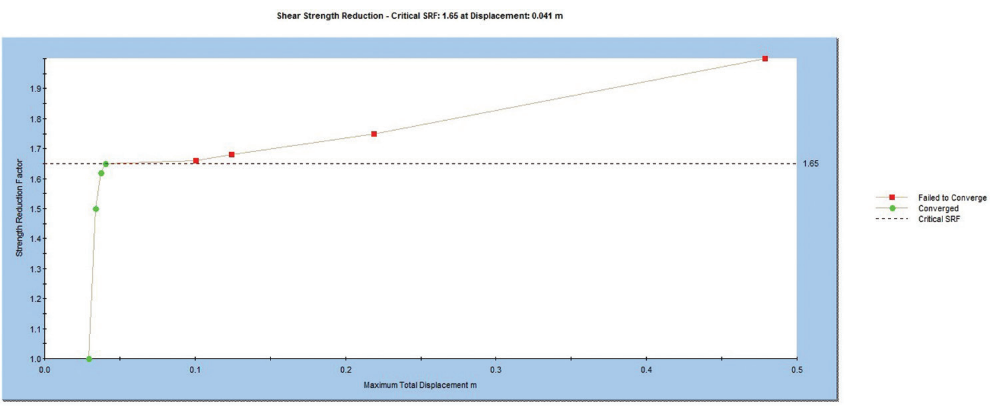


Fig. 5.14: Horizontal displacement (Left side) and Total displacement (Right side) for stabilised slope under seismic condition

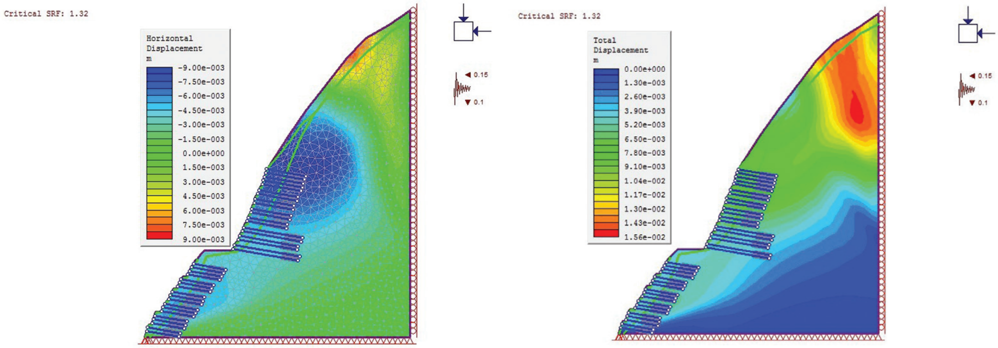


Fig. 5.15: Variation of strength reduction factor with maximum total displacement for seismic case (with stabilisation)

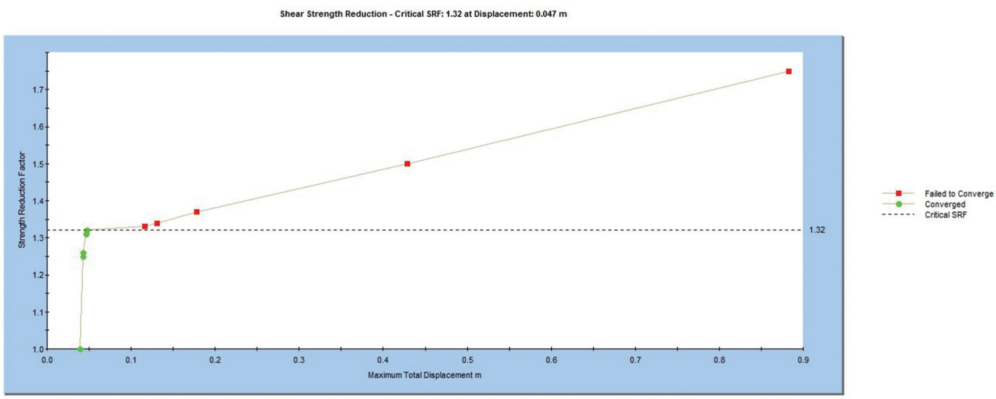
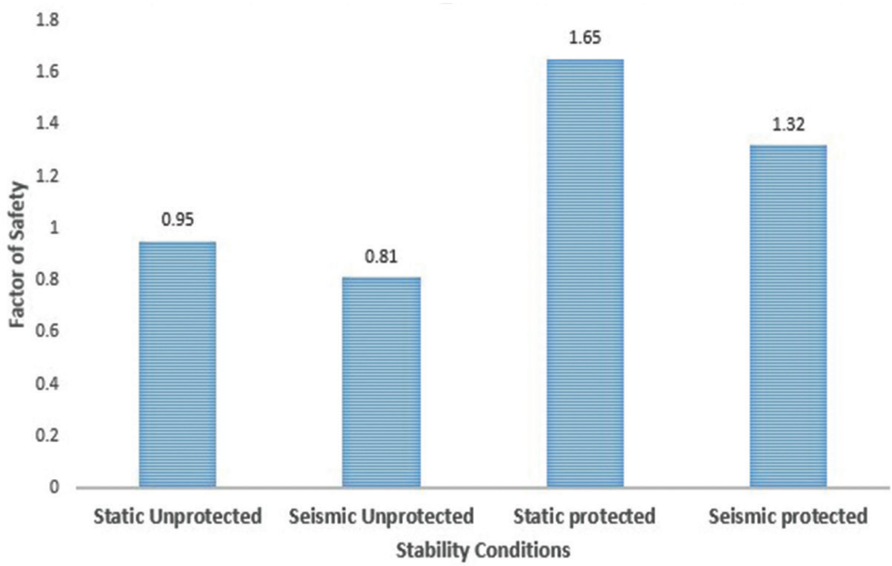


Table 5.3: Comparative Table between Protected and Unprotected Stability

Parameter	Static Case		Seismic Case	
	Unprotected	Protected	Unprotected	Protected
F.O.S	0.95	1.65	0.81	1.32
Max. Displacement (m)	2.6	0.041	2.8	0.047

Following chart shows the relative increase in critical SRF values for stabilised and unstabilised conditions under the static and seismic case.

Fig. 5.16: Variation of strength reduction factor under different stability conditions



Discussion

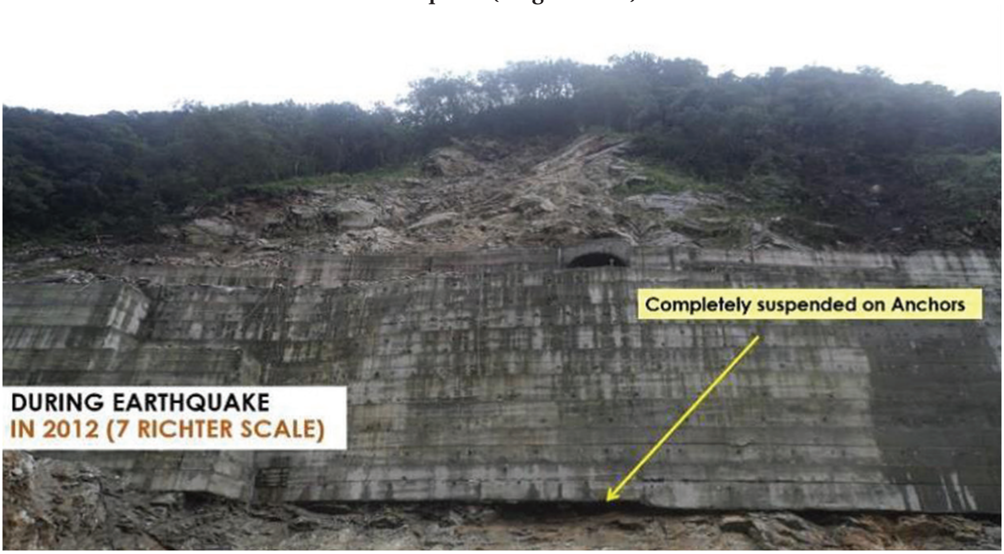
Stabilisation of poor class rock in seismic and region of high pore pressure is very vital. Use of prestressed cable anchor provides an effective and long term solution. It is clear from the above finite element analysis results that there is a significant improvement in the critical safety factor results. It has been observed that there is 73.68 per cent and 62.96 per cent improvement in safety factor values under static and seismic conditions respectively after inclusion of prestressed cable anchors. Displacement is also a critical criteria for slope stability. In the present case, displacements restrained within the nominal limits for the static case and seismic case.

The mechanism behind the stabilisation is due to increase in the shear strength along the failure surface, as pretension cable anchor increases the active resistance resulting from the stage of mobilisation of sheared mass along the slip line.

Conclusions

The present analysis shows that there is a significant improvement in the safety factor results. The current status of the project site is that the 50 m vertical cladding wall having prestressed cable anchor with no base support survived the 2012 earthquake of 7 Richter magnitude scale with no signs of distress.

Fig. 5.17: Condition of stabilised slope with prestressed cable anchor after 2012, earthquake (Magnitude 7)



References

BS 8539: 2012 Code of practice for the selection and installation of post-installed anchors in concrete and masonry.

Gupta, S. (2006). “Design of prestressed cable anchor for stabilization of the Himalayan slopes”. Dissertation, IIT Roorkee.

Hoek, E. and Brown, E.T. (1997). “Practical estimates of rock mass strength”. *International Journal of Rock Mechanics and Mining Sciences*, 34(8):1165-86.

Hoek, E., Carranza-Torres, C.T. and Corkum, B. (2002). “Hoek-Brown failure criterion”” 2002 edition. In Proceedings of North America Rock mech. soc. http://www.rocscience.com/library/pdf/RL_1.pdf. Accessed on October 6, 2011.

IS 14268: 1995 Uncoated stress relieved low relaxation seven-ply strand for prestressed concrete by Bureau of Indian Standards (BIS), New Delhi.

IS 14448: 1997 Code of practice for reinforcement of rock slopes with plane wedge failure by Bureau of Indian Standards (BIS), New Delhi.

IS 1893 (Part 1): 2016. Criteria for Earthquake Resistant Design of Structures, Part 1: General Provisions and Buildings by Bureau of Indian Standards (BIS), New Delhi.

Lees, A. (2001). *Geotechnical Finite Element Analysis*. ICE Publishing, One Great George Street, Westminster, London.

Muhunthan, B. et al., (2005). “Analysis and design of wire mesh/cable net slope protection”. Technical report no. WA-RD 612.1, University of Washington.

Pandit, K., Sarkar, S., Godayal, A. and Shazan, M. (2019). “A FEM Study on Bearing Capacity and Safety of Shallow Footings on Slopes for Garhwal”. ISRM 14th International Congress on Rock Mechanics and Rock Engineering, Foz do Iguassu, Brazil. (https://tinyurl.com/yy64fuq3) DOI: https://doi.org/10.1201/9780367823184

Rocscience (2012). “A 2D finite element program for calculating stresses and estimating support around the underground excavations”. Geomechanics software and Research, Rocscience INC, Toronto, Canada.

Technical Report: An Assessment of dams in India’s North East seeking carbon credits from clean development mechanism of the United Nations framework convention on climate change.